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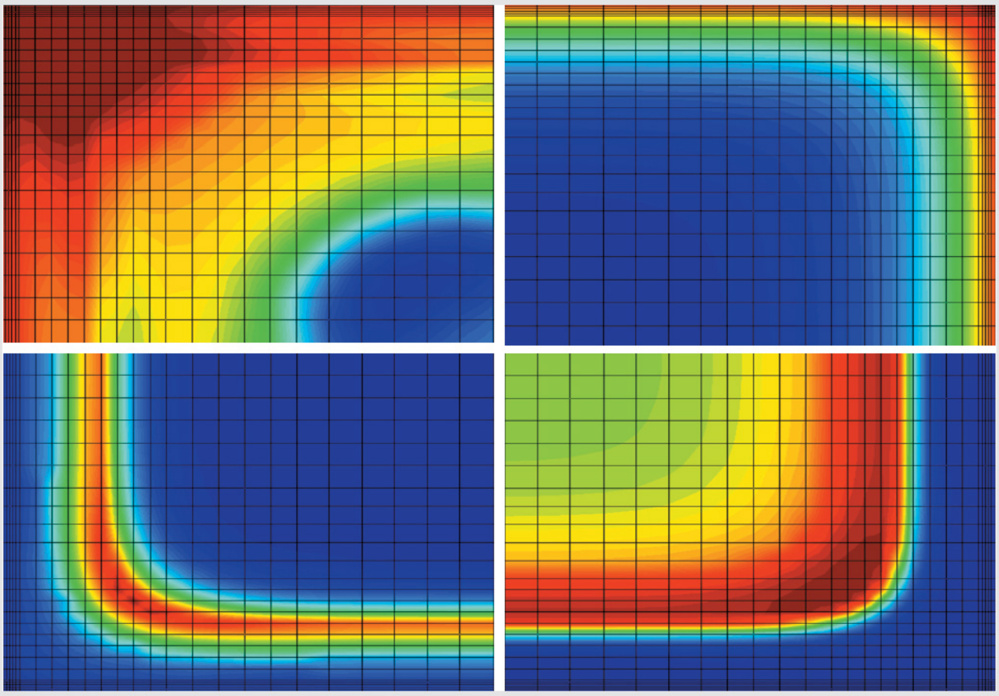

Fire design of concrete structures – materials, structures and modelling

state-of-art report

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State-of-art report prepared by

Working Party 4.3-1

April 2007

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category	minimum approval procedure required prior to publication
Technical Report	approved by a Task Group and the Chairpersons of the Commission
State-of-Art Report	approved by a Commission
Manual, Guide (to good practice) or Recommendation	approved by the Technical Council of *fib*
Model Code	approved by the General Assembly of *fib*

Any publication not having met the above requirements will be clearly identified as preliminary draft.

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Cover images: Quarter column section distribution of damage, temperature, relative humidity and vapour pressure (see Figure 2.8)

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Preface

In recent years fire design of concrete structures has emerged as a topical and mature subject of high profile and even emotive interest to both experts and the public. The high profile is largely prompted by severe damage to concrete in a number of recent tunnel fires, whilst subject maturity is due to the considerable amount of research and development that has taken place to date world-wide.

The result of this international research should be presented to a wider group of concrete professionals who are conscious of the serious lack of guidance in this field. For this reason the Federation International du Béton - *fib* (International Federation of Structural Concrete) took the initiative to publish this Bulletin in order to meet this demand within the framework of the fib Task Group 4.3 "Fire Design of Concrete Structures" under Commission 4 "Design of Concrete Structures".

The present Bulletin is a result of the efforts of Working Party 4.3.1 headed by Professor Gabriel Alexander Khoury, who has specialised in this topic for some 33 years. Professor Khoury has supplied the results of his many years of research and has written the majority of the report incorporating the outcome of a number of international research projects in this field. Also a number of other authors have contributed to this state-or-the-art bulletin, and the Task Group has carefully reviewed and commented on the report.

Significant advances have already been made to allow practicing engineers to carry out better design of concrete and concrete structures against fire. The publicaton of this report is, therefore, timely and will hopefully be of major benefit to the concrete industry.

Niels Peter Høj
Convener of *fib* Task Group 4.3

Contents

1 Introduction

This publication is intended to provide guidelines rather than just data on the influence of fire on concrete and concrete structures. Therefore, the data published herein are a selected number for the purpose of supporting interpretation and the flow of concepts.

The guidelines presents a general brief outline of the effect of fire on both concrete material and concrete structures with emphasis being placed upon the important developments in the past few years, namely: (a) the increasing use of high strength concrete (HSC) in buildings, tunnels and bridges; (b) the growing acceptance of the use of performance based fire engineering calculations for the structural analysis and design against fire; (c) the problem of, and solutions to, explosive spalling; and (d) fires in tunnels - an issue of very high profile in recent years.

This report is not intended to be an exhaustive review of the effect of fire on concrete and concrete structures, nor present a database of properties at high temperature. There are numerous publications on the subject - covering both materials and structures - in journals, conference proceedings, books [BAZ 96], [ANC 86], [SHI 87], [BUC 01] and the work of the RILEM Committees [SCH 86], [SCH 90]. Instead, the main aims of this document are to present recent trends and developments, highlight key influencing factors, bring together holistically the disparate but related issues in one short document, highlight the deficiencies in current practice and point to the future. The basic principles of performance based codes and fire engineering are also presented on the assumption that the reader is not a specialist in this field.

1.1 Background

Investigation into the effect of fire on concrete and concrete structures has been conducted since at least 1922 [LEA 22] primarily in relation to buildings. Up to about a decade ago, fire research was focused upon the behaviour of *normal strength* concrete at high temperatures, and engineers mainly employed *prescriptive* methods of design to ensure structural stability in fire for a sufficient period of time to allow people to escape and fire services to extinguish the fire. Since then, there have been two major developments in this field, namely: (a) the increasing use of new types of concrete such as self compacting concrete and high strength concrete in buildings, tunnels and bridges allowing the use of more slender and complex structures but with a higher tendency to experience spalling in fire, and (b) the growing acceptance of the use of performance based fire engineering calculations for the structural analysis and design against fire.

Damage to HSC in recent high profile tunnel fires in Europe has provided impetus for research into improving: (a) tunnel safety, and (b) concrete performance in fire. The departure from traditional concrete mixes and structures requires a design based on scientific principles, rendering inappropriate simple extrapolation of existing practice. Traditionally, concrete has been regarded as "fireproof" because of its incombustibility and its high thermal insulating properties. The three main problems experienced by traditional concrete in fire, namely (a) deterioration in mechanical properties, (b) damage caused by thermal deformations, and (c) spalling can be addressed by careful mix design and structural design. Recent tests have shown that concrete containing the appropriate type and amount of polypropylene (PP) fibres (*henceforth PP fibres*) will significantly reduce the risk of spalling even when subjected to a tunnel fire that is more severe than an ISO 834 or a hydrocarbon fire. In fact, sprayed concrete containing PP fibres is used as a fire barrier to protect tunnel linings. Careful design can also produce concrete with improved mechanical properties for temperatures as high as 300-600°C. Research to date has shown that concrete is a versatile material and, if appropriately designed, can be inherently fire resistant. This has led to the current era of "bespoke" fit for purpose concrete mix design against fire, but which also holistically serves the structural and architectural function of the material.

At the structural level, the impetus for the development of fire engineering assessment methods came from the limitations inherent in the traditional prescriptive methods of design. Several countries have already developed performance-based codes (e.g. U.K., Sweden, Norway, New Zealand and Australia) and many more countries are in the process of doing so. This topic is currently the focus of much discussion, research and development worldwide. This has led to new ideas for improving fire safety, thus encouraging the engineer to develop new creative solutions [AND 98]. Traditionally fire safety was a set of conventions rather than a rational approach with engineering tools. To move forward it is essential that the whole package of conventions, and requirements, are re-examined in a holistic and scientific manner.

1.2 Fire scenarios

1.2.1 Real fires

The response of concrete material and structure to a real fire depends upon the nature of the fire which can vary considerably from fire to fire. Moreover, the gas temperature will vary throughout the fire compartment. The key feature of the fire relevant to the concrete structure is the temperature-time curve imposed by the fire at its surface (Figure 1.1), the key parameters of the which are: (a) The ***heating rate*** (i.e. the rate of temperature increase) which influences the development of temperature, moisture and pore pressure gradients within the concrete structure, (b) The ***maximum temperature level*** which influences the nature of the physico-chemical relations in the material and through this its properties; (c) The ***duration*** of the fire which influences the temperature development in the structure with time; and (d) the ***cooling regime*** (e.g. water cooling would have a different influence upon the material and the temperature distribution from "natural" cooling). The heating rate is accelerated during flashover (Figure 1.1) which could be life threatening (Figure 1.2).

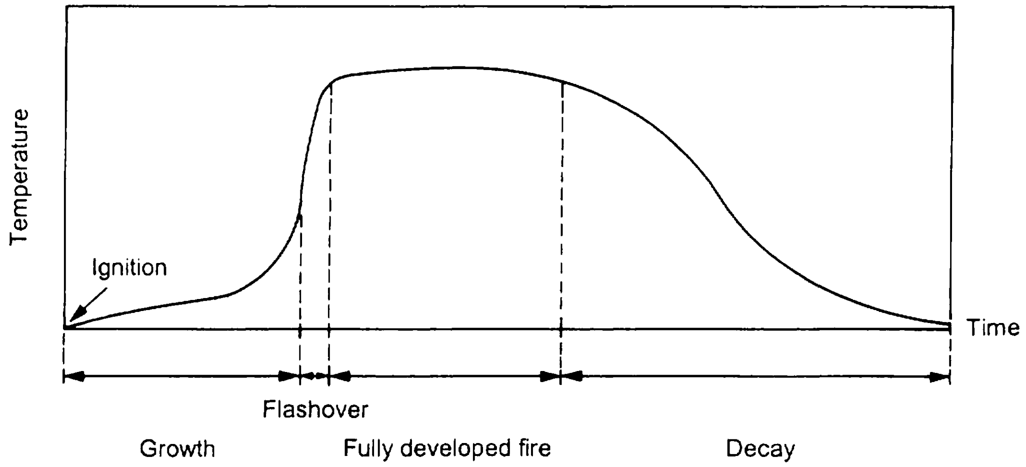


Figure 1.1: Temperature development stages in a "real" fire for enclosed spaces < $500m^3$ [COX 02].

When assessing the fire resistance of a concrete structure, and irrespective of the method of assessment used (either experimentally or theoretically) the first step is to model the real fire to a realistic and conservative fire scenario. Such models fall into three categories: (a) Nominal fire curves; (b) Parametric fire curves; and (c) Multi-zone models.

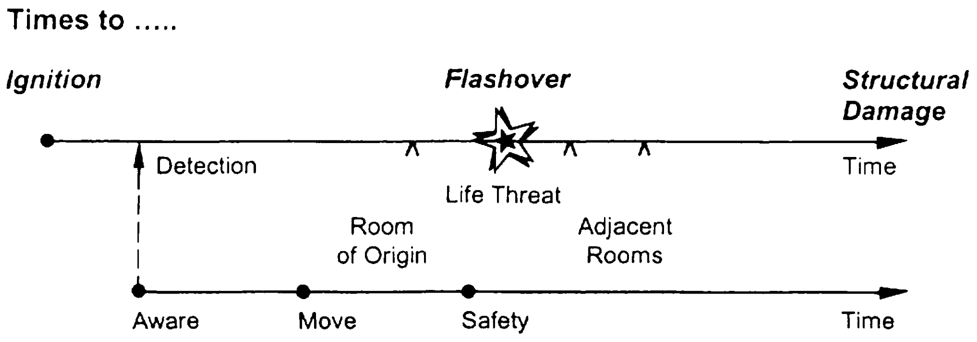


Figure 1.2: Hazard fire development clock [COX 02]

1.2.2 Nominal fire curves

Nominal curves are idealised simplified fires represented by a temperature-time relation as shown in Figure 1.3 (Table 1.1). No account is taken of the influences upon temperature development and distribution of such factors as the amount of combustibles materials etc. Moreover, nominal fire curves (Figure 1.3) assume a uniform gas temperature throughout the fire compartment. The temperature-time curves in "Standard" fires used in testing, analysis and design were established from experience in real fires and fall into three broad categories depending upon the application (i.e. Buildings, offshore/petrochemical, tunnels) as explained below:

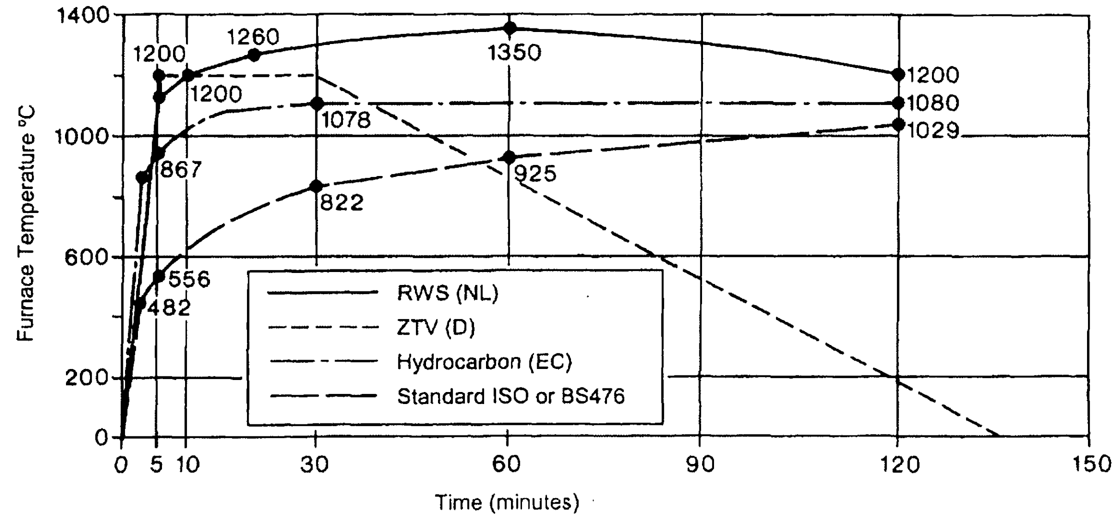


Figure 1.3: Examples of a few Standard fire scenarios for buildings (ISO834 or BS476 (or other national applications of ISO 834)), offshore & petrochemical industries (Hydrocarbon), and tunnels (RWS & ZTV).

Note 1. Many other design fire curves exist which include the modified hydrocarbon curve (HCM).
Note 2. Design curves vary in heating rate, maximum temperatures and cooling (or not) branches.
Note 3: These idealised fire curves were established from experience in real fires which vary considerably from fire to fire. In reality all fires have a cooling phase.

1.2.2.1 Buildings

The Standard Furnace nominal fire is by far the most commonly used fire scenario to assess the fire resistance of structures in buildings and is given by the standard fire curve according to ISO 834 [ISO 75]. Most tests and calculation methods are based on the standard fire curve. It represents a typical building fire based upon a cellulosic fire in which the fuel source is wood, paper, fabric etc. It does not include a final decay stage but the gas temperature endlessly increases, whereas in a real fire the temperature eventually will decrease once most of the combustible materials have been burnt. Temperature increases from 20°C to 842°C after the first 30 minutes. This fire profile has a slow temperature rise up to 1,000°C over a period of 120 minutes

1.2.2.2 Offshore and petrochemical industries

In the 1970s, the oil company Mobil investigated Hydrocarbon (HC) fuel fires [EC1 91] and developed a temperature-time profile with a rapid temperature rise in the first 5 minutes of the fire up to 900°C and a peak of 1,100°C. This research laid the foundation for test procedures to assess fire protecting materials for the offshore and petrochemical industries. This fire curve represents a heat load of 200 kW/m^2. During the 1990s, HC curves were developed for heat loads of 50-300kW/m^2 to represent a wider range of HC fires.

T-t curve	**Normal Application**	**Heating Rate**	**Max. T. (°C)**	**Cooling Branch**	**Duration (mins)**
ISO 834 standard fire	Buildings (now also suggested for tunnels)	821°C after 30 mins (average heating rate of ≈ 30°C/min), 925°C after 60 mins, 1,000°C after 2 hrs & 1,150°C after 4 hrs	None but 1,000 after 2 hrs	No	No limit
Hydrocarbon (Eurocode 1) Represents 200kW/m^2 heat flux	Petro-chemical/ Offshore	900°C after 5 mins (average heating rate of ≈ 170°C/min) and 1,100°C after 45 mins	1,100	No	No limit
ZTV (D)	Tunnels	1,200°C after 5 mins (average heating rate of ≈ 240°C/minute)	1,200	At 30 mins (55 for rail) start of 110 mins of linear temp decay	170 mins including decaying branch
RWS (NL) (300MW Fire)	Tunnels	1,100°C after 5 mins and 1,350°C after 60 mins	1,350	Reducing from 1,350°C to 1,200°C after 120 mins	120
Modified Hydrocarbon Represents 345kW/m^2 heat flux	Tunnels/ Offshore	1,200 °C in less than 10 mins and 1,300°C about 20 mins later	1,300	No	No limit - but expected < 120mins

Table 1.1: Nominal temperature-time (T-t) curves

1.2.2.3 Tunnels

A spate of major tunnel fires have indicated an even more severe fire scenario needs to be considered. The Netherlands established the "RWS" fire curve for the evaluation of passive protecting materials in tunnels [RWS 98] representing a most severe hydrocarbon fire rapidly exceeding 1,200°C and peaking at 1,350°C (melting temperature of concrete) after 60 minutes and then falling gradually to 1,200°C at 120 minutes - the end of the curve. RWS is intended to simulate tankers carrying petrol in tunnels with a fire load of 300MW causing a fire for 2 hours and was established on the basis of Dutch experience in model tunnel fires. The RABT German fire curve (also referred to as the ZTV-fire) comprises a descending branch and represents a less severe fire scenario in tunnels than the RWS curve reaching a maximum temperature of 1,200°C (melting point of some aggregates) sustained up to 1 hour before decaying to ambient. A new "modified" or "increased" hydrocarbon curve is being proposed instead of both RWS and RABT curves. This proposed modified HC-curve represents a heat load of 345 kW/m^2 to simulate a severe fire in a tunnel. The temperature rises to 1,200°C in less than 10 minutes and reaches a maximum of 1,300°C after about 20 minutes. Such a severe fire is not supposed to last more than 120 minutes.

1.2.3 "Natural" fires - Parametric curves in buildings

"Natural" fires or parametric fire curves used for building compartment fires (Figure 1.4) are described by a full set of time-temperature curves available for different fire loads (25-3750 MJ/m^2) and opening factors (0.01-0.3 m$^{0.5}$). Natural fires or parametric fire curves give an improved prediction of a real fire compared to the nominal fire curves when the fire compartment has a floor area of less than 500 m^2. Parametric curves still assume a uniform temperature distribution throughout the fire compartment. However, the actual temperature development in time depends on additional parameters, such as the amount and type of combustibles, ventilation (opening factor), thermal inertia of enclosing walls and floors etc. Parametric fire curves typically exhibit both ascending and descending branches in the temperature-time curve. Parametric curves are not very practical for testing. It may be difficult to control the test furnaces to simulate the cooling branch. However, parametric curves are very suitable for calculation. They offer the ability to improve the accuracy of the predicted behaviour of a fire exposed structure compared with the use of nominal fire curves. As a result, the design can be more economical. However, it requires an adequate constitutive model of the structural materials under cooling conditions. Especially for concrete, such a constitutive model is not simple.

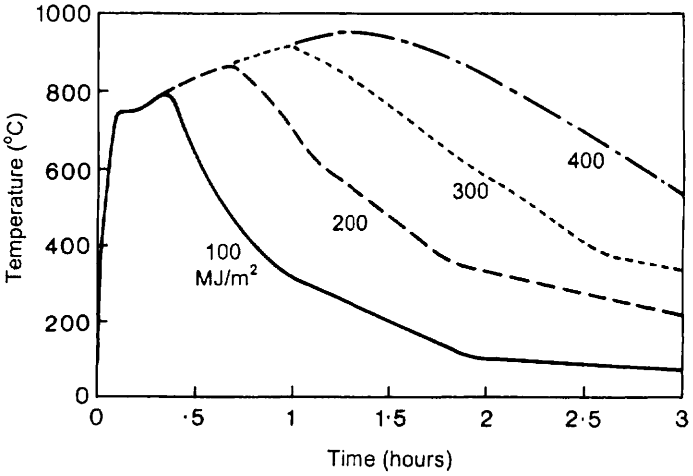


Figure 1.4: Time temperature development of natural (parametric) fires with cooling branches for heat loads ranging from 100 to 400 MJ/m^2 and an opening factor of 0.04m$^{0.5}$ [MAG 70].

1.2.4 Multi-zone & CFD models

Even more advanced than the parametric fire curves are the multi-zone and computational fluid dynamics (CFD) models. The models solve the equations of the conservation of energy and mass for each zone or element. These are mainly used for research purposes. Two-zone models (e.g. Ozone) are used in various countries (e.g. Luxembourg, France, Sweden) at the design stage and assume a constant temperature in the smoke layer, while CFD models simulate a more realistic scenario of spatially varying temperature. To date CFD has hardly been used at the design stages - but this is changing rapidly. These models, especially the CFD, are employed primarily to study the dispersion of smoke and heat but are rarely used to assess the impact of the fire in terms of heat fluxes into structural elements.

2 Assessment of thermal/structural response to fire

In addition to prescriptive and testing methods, technical development to date allows the assessment of thermal/structural response to fire by calculation. The methodologies, both by testing and by calculation can vary considerably in complexity and, therefore, cost. It is necessary to ensure an acceptable cost-benefit solution in engineering practice, though not always in research. This section is devoted to these complex issues which are presented holistically.

2.1 Background

The basis of most current legislation is the traditional "standard" fire test, in which a building element, separated from its built environment, is placed in a furnace and exposed to a prescribed standard (ISO 834) fire. The requirement that is set is for the building element to withstand this thermal load for specific periods of time, selected more or less arbitrarily in relation to the time needed for people to escape from buildings. Distinction is made as to the type of building involved. Primarily, the requirements are set with a view to ensuring the safety of occupants, "neighbours" and the fire brigade, and address issues pertaining to public law. As a secondary consideration, pertaining private law, additional requirements may be set, to protect investments and minimise economic losses. It is apparent, therefore, that most existing building codes are predominantly of a prescriptive nature, i.e. the codes provide (prescribe) solutions or give (prescribe) analytical tools to check solutions, which fulfil the requirements. Furthermore, most legislation is based on the prevention concept which is in turn based on compartmentation. This means that requirements are set to maintain the fire for a certain period of time in the compartment where it started for a sufficiently long period to allow occupants to escape and the fire brigade to intervene. These time periods are normally expressed in minutes and increase in 30 minute intervals, ranging typically from 30 to 120 minutes. Recently, as a result of the integration of the various fire disciplines, a more advanced methodology has emerged in the form of Fire Safety Engineering which in its most developed form assesses realistically the response of the actual structure, including that beyond the fire compartment, when exposed to a realistic fire. This method is also called "performance" based as opposed to the "prescriptive" method mentioned earlier. Prescriptive fire resistance requirements vary considerably throughout Europe as indicated in Figure 2.1.

2.2 Fire resistance

2.2.1 Requirements

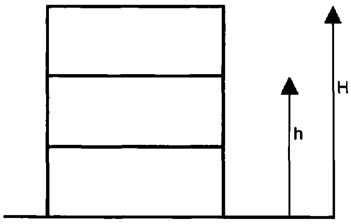
Fire causes heat to flow into the concrete structure. The temperature in the concrete mass will rise, causing thermal expansion of the constituents, evaporation of moisture, build up of pore pressures and degradation of the mechanical properties. The designer should ensure that all these factors combined do not prejudice the structure's primary separating and load bearing functions. The fire design of concrete structures, therefore, aims to fulfil the separating function and/or load bearing function of the structure for a given duration of time. In load bearing members (e.g. beams, columns, walls and slabs), the load bearing function prevents local and potentially wider progressive structural collapse in fire. In general, the separating function is relevant only for structures that are part of the fire compartment envelop such as walls and slabs.

Fire resistance is defined in Eurocode 2 [EC2 05a] as the ability of a structure, a part of a structure or a member to fulfil its required functions (load bearing function and/or separating function) for a specified fire exposure and a specified period of time. The fire resistance is measured in minutes up to failure for nominal fire curves. It will be shown below that the assessment of the fire resistance requires some simplifications, rendering measurements in individual minutes pointless. Therefore, fire resistance is typically expressed in classes of 30 minute intervals.

Minimum Periods (minutes) for elements of structure

in the following building types								according to the regulations of										
Building type	n	h	H	X	L	b	x(*)	S	B	CH	D	F	I	L	NL*4	FIN	SP	UK
Industrial Hall	1	0	10	20	100	50	2	YES	0	0	0	30 *2	0/60 (7)	0	0	0	-	0 *1
								NO	0	0	(1)	30 *2	30/90 (7)	0-60	0	0	-	0 *1
Commercial center and shop	1	0	4	500	80	80	4	YES	0	0	0	0 H 30 V	60/90 (7)	30	0	0	90	0 *1
								NO	(1)	(1)	(1)	30	90/120 (7)		0	30	90	0 *1
Dancing	2	5	9	1000	60	30	4	YES	0	0	(2)	60	(8) (9)		0	60 (4)	90	30
								NO	0	30 *3	90	60	60	30	0	60 (5)	90	60
School	4	12	16	300	60	20	4	YES	60 (6)	0 30	(2)	60	(8) (10)		90 (13)	60 (4)	60	60
								NO	60 (6)	0-30 *3	90	60	60	90	90 (13)	60 (5)	60	60
Small rise Office Building	4	10	13	50	50	30	2	YES	60 (6)	0 30	(2)	60	(8) (9)		90 (13)	60 (4)	60	30
								NO	60 (6)	60	90	60	60	90	90 (13)	60 (5)	60	60
Hotel	6	16	20	60	50	30	2	YES	60 (6)	30-60 *3	(2)	60	(8) (11)		120 (13)	60 (4)	90	60
								NO	60 (6)	60	90	60	60	90	120 (13)	60 (5)	90	60
Hospital	8	24.5	28	60	70	30	2	YES	120	60	(2)	60	(8) (12)	90/120	120 (13)	60 (4)	120	90
								NO	120	90	90	60	120	120	120 (13)	60 (5)	120	90
Medium rise Office Building	11	33	37	50	50	30	2	YES	120	60 *3	(2)	120	(8) (9)	90	120 (13)	120 (4)	120	120
								NO	120	90	90	120	90	120	120 (13)	120 (5)	120	(3)
High rise Office Building	31	90	93	100	50	50	2	YES	120	90	90	120	(8) (9)	120	120 (13)	120 (4)	120	120
								NO	120	90	(3)	120	120		120 (13)	120 (5)	120	(3)

n = Number of storeys, ground level included
h = Height of top floor above ground
H = Height of the roof above ground level
X = Number of people to be evacuated by storey
L = Length of the compartment
b = Width of the compartment
x = Number of exit routes (* indicate your requirement, in case of no requirement the values beneath may be considered)
S = Sprinkler



(1) compartment size too large
(2) no regulation adopted
(3) not allowed
(4) q > 600 MJ/m² floor
(5) q < 600 MJ/m² floor
(6) new buildings + extension or structural charges of existing buildings
(7) Periods usually required by local authority (there's nt still national regulations)
(8) Sprinkler is a possible alternative to other requirements (case by case by authority)
(9) Required for q > 920 MJ/m²
(10) Required in underground rooms for q > 550 MJ/m²
(11) Required over 1000 beds
(12) Required over 300 beds
(13) Required for q_{perm} > 500 MJ/m²; otherwise the requirement may be decreased by 30 min.

*1 Roof structure & structure only supporting roof requires no fire resistance. Therefore single storey building structure normally has no requirement.
*2 If H < 10m : no requirements (R0)
*3 To be checked by SIA Doc. 81
*4 For main load bearing structure and if no escape routes are blocked due to failure, otherwise the minimum requ. is 30 min.

For FINLAND:	for load bearing structures, not for separating structures
For FRANCE :	H = horizontal roof structure V = column
For NETHERL.:	floor area fire compartment is generally limited to 1000 m2

Figure 2.1: Overview of prescriptive fire resistance requirements throughout Europe indicates considerable variation.

2.2.2 The load bearing function

The load bearing function is simply limited by collapse under applied loads. With a view to protecting furnaces and measuring equipment, deflection criteria are set for practical reasons. All relevant failure modes should be evaluated, such as bending failure, shear failure, buckling failure and anchorage failure. With parametric fire curves [MAG 70] both the heating and a subsequent cooling phase are considered. The minimum load-bearing capacity occurs during the cooling phase and consequently the risk of collapse is then greatest. When bending failure governs the load bearing behaviour, the minimum load-bearing capacity is reached when the reinforcing steel temperature is at its maximum value - some considerable time after maximum surface temperature has been attained.

2.2.3 The separating function

The separating function is assessed using two criteria: integrity and insulation. The integrity criterion sets a limit to ensure that the structure does not allow the penetration of flames and hot gasses through it as a result of gaps or cracks. The insulation criterion sets a limit, in nominal fires, for the temperature rise at the unexposed side of the structure to 140°C on average or 180°C at local peaks. For real, or parametric fires, the corresponding limits of temperature are 200 and 240°C respectively (assuming same duration of fire for both curves) because the real fire contains a cooling phase. These are not scientific limits but ones used for practical purposes essentially to improve safety.

2.2.4 Requirements

Hence, normally, each part of the structure will have a different function during a fire according to its type and position. This function could be to contain a fire (as with non load-bearing walls), to support the design loads (as with beams and columns), or both (as with a floor). The fire design process for separating structures only comprise thermal analysis, whilst for load-bearing structures both thermal and structural analyses are required.

Meeting the requirements, therefore, results in a design with:

- overall dimensions of the section of an element sufficient to keep the heat transfer through this element within acceptable limits; and
- average axis distance of the reinforcement sufficient to keep the temperature of the reinforcement below critical values which are 500°C for carbon reinforcing steel and 350°C for prestressing steel at a load utilisation of 100%. If the load utilisation is less, then the critical temperatures can be increased as defined in Eurocode 2.
- mix design to ensure: (a) relatively high thermal stability of the concrete (e.g. appropriate thermally stable aggregates); (b) low probability of spalling (e.g. use of polymer fibres in humid tunnels and in high strength concrete; (c) low thermal and parasitic stresses during heating/cooling (e.g. use of low thermal expansion aggregates).
- Thermal barrier protection when required to reduce heat flow into the concrete and mitigate the effects of pore pressures and thermal stresses.

The fire resistance of a whole concrete structure would not necessarily be that ascribed to its individual elements. It could be better or worse.

Better fire behaviour could arise from such factors as robustness, adequate continuity of reinforcement, reduced level of loading, composite construction, load sharing mechanisms and the availability of alternative paths for load support. The provision of continuity of reinforcement in the design allows the redistribution of forces and moments to gradually take place towards the parts of the structure not exposed to the fire as the exposed parts are weakened, thus ensuring an improved fire resistance relative to the situation of the simply supported single element.

Worse fire resistance could arise from loads introduced by the thermal deformations, acting also on unexposed parts of the structure. For instance, columns can shear off as a result of the thermal expansion of a fire exposed floor, (Figure 2.2). These thermal loads can cause deterioration in the concrete structure, such as cracks, which lead to a reduced load-bearing capacity. If the loaded parts do not provide sufficient deformation capacity, premature failure can occur. Also, sufficient rotation capacity is required at intermediate supports of continuous slabs in order to allow for a full redistribution of the bending moments up to the plastic capacity in the field.

It is, therefore, necessary to pay particular attention to detailing for reinforcement and building layout.

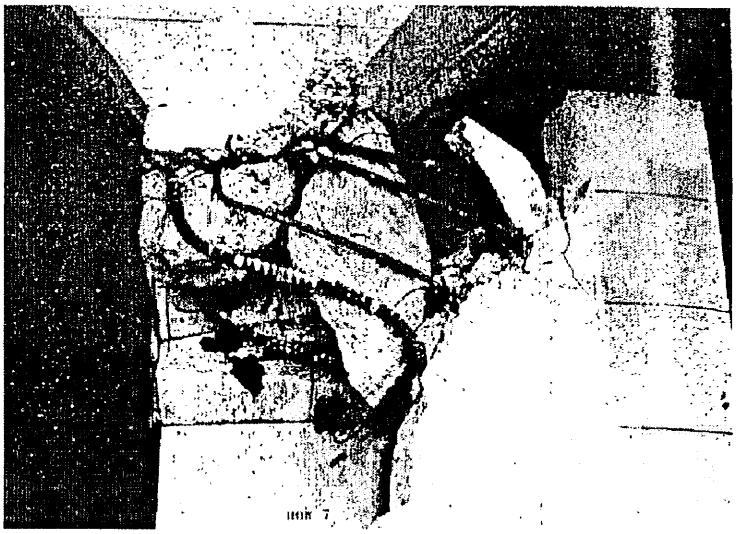


Figure 2.2: Example of a shear failure in an unexposed concrete column due to the thermal expansion of the fire exposed floor connected to it [BEI 02].

2.3 Scope of assessment

The scope of the assessment is defined by the fire and structural models used. This is illustrated in Figure 2.3 as a matrix of increasing complexity of each. At the simplest level only a single structural member is assessed exposed to a nominal fire scenario, while at the most complex, a whole structure is assessed exposed to a multi-zone (or real) fire scenario. As for the method of assessment this could be either by means of testing or calculation or both, again with increasing levels of complexity. The traditional use of nominal fire curves fits more into the conventional approach, while multi-zone and CFD models fit more into the rational approach to fire safety (Table 2.1).

2.4 Levels of structural assessment

According to Eurocode 2, a structure can be evaluated at three levels of increasing complexity and size: (a) member analysis, (b) substructure analysis and (c) global structural analysis (Figure 2.3).

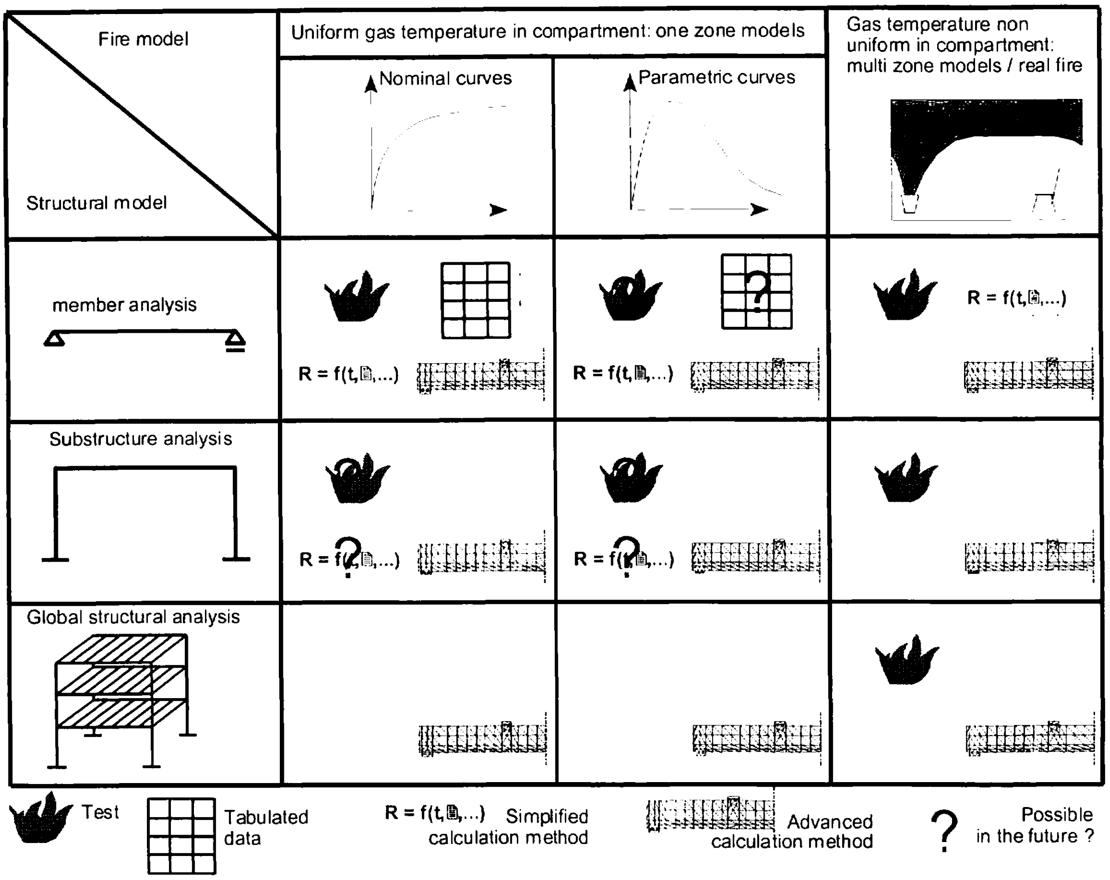


Figure 2.3: Overview of fire assessment methods for increasing complexity of fire and structural models.

2.4.1 Single member assessment

If the fire resistance is assessed experimentally, the testing facilities normally limit the size and complexity of the structure to be assessed. Therefore, tests are generally carried out on individual members such as single span beams, slabs and columns. Member analysis also allow for the use of tabulated data and calculation methods (often simplified). Due to the forces that can develop as a result of thermal expansions from fire exposure, it is essential to apply boundary conditions that are similar to the ones expected for the structural element in the total structure. However, these restraining forces will generally vary during the fire. Therefore, the proper choice of the boundary conditions in either the test or in the calculation is not straight forward. In Europe, the choice was made to refrain from testing under restraint conditions, however in the USA, testing under restraint conditions is adopted in the legislation. As a consequence of the simplifications of the boundary conditions, there can be a huge difference between the actual fire behaviour and the one obtained from the experiment or the calculation.

2.4.2 Substructure assessment

As explained above, the difficulty and weakness of single member analysis lies in the appropriate choice of the boundary conditions. In order to overcome this problem, analysis could be performed at the level of a substructure. Eurocode 2 [EC2 05a] states that the global structure should be divided into substructures in such way that the interaction of thermal expansions and deformations during fire can be approximated by time independent support and boundary conditions. The substructure, therefore, might extend beyond the boundaries of the fire compartment. Although this approach is more accurate than single member analysis, it still is not easy to determine the positions in a structure where a subdivision can be assumed, nor is it easy to transform the support and boundary conditions at these points into adequate loading and restraint conditions for the substructure. Generally, the size of a substructure exceeds the capabilities of standard fire test furnaces and therefore, tests on substructures are very rare.

2.4.3 Global structural assessment

The most accurate method of predicting structural response is through the assessment of the whole structure by full-scale testing or modelling. Redistribution of forces and the effect of restraint thermal expansions can adequately be taken into account. The main drawback is obviously the complexity and cost of this approach. As a result their use is limited.

2.5 Methods of assessment of fire resistance

The fire resistance of concrete structures can be assessed by one of the following five distinct methods:

- Standard fire tests
- Tabulated data (largely prescriptive but also increasingly based on calculations)
- Simplified calculations, neglecting complex effects such as thermal stresses.
- Advanced calculations (largely performance based)
- Full scale fire tests

Obviously, also combination of fire tests and calculations can be used, for instance to extend the field of application obtained in a test by calculation.

Fire tests can range from the standard fire test on single elements to the full-scale testing of a whole concrete structure in fire (e.g. building or tunnel). Calculation methods can also range from simplified methods to the most complex thermo-hydro-mechanical finite element analysis (Figure 2.4)

For decades, standard fire testing and the use of tabulated data have been the main means for determining fire resistance. These are primarily prescriptive methods. Furthermore, testing is expensive and the results cannot easily be extended to structures with alternative geometry, loading arrangement or boundary conditions. Therefore, there has been a strong ongoing development over the last decades to determine the fire resistance theoretically by means of engineering calculations. These are primarily performance based methods that are being accepted in an increasing number of countries.

The integrity criterion can generally be met be providing adequate detailing, i.e. reinforcement between members, sufficient dimensions, etc. The criteria for insulation and load bearing must be met with one of the five assessment methods listed in Table 2.1.

	Assessment Method	**Complexity & Accuracy**	**Prescriptive Conventional**	**Performance based**
1	Standard testing	△	▽	△
2	Tabulated data			
3	Simplified calculations			
4	Advanced calculation			
5	Full-scale testing			

Table 2.1: Assessment methods listed in increasing complexity and accuracy

2.5.1 Fire testing

Fire testing of an element or sub-assembly is a more expensive option when compared with using tabulated data or calculations - particularly for larger more complex structures. Any form of concrete element covered by a valid fire test report may be deemed to have the fire resistance ascribed to it by such a test provided that the element has similar details of construction, stress level, and support as the test specimen. Standard tests for fire resistance are usually conducted on single building elements where it is not feasible to reproduce in the test furnace the nature and magnitude of restraint and continuity of the adjoining construction. In some cases, therefore, the fire performance of structural elements in the building could be expected to be much greater than that of the simple element when tested in the furnace. In other cases, thermal movement can reduce the fire resistance. An advantage of fire testing over prescriptive methods is that it provides an indication of temperature distributions within, and deflections of, the element during heating as well as detailing weaknesses not discovered without tests. However, its accuracy is sensitive to the testing apparatus and method employed - hence the considerable discussions at international committees level on harmonisation of laboratory testing between the different countries. The preparation for testing and performing the test is lengthy and the cost of its setting-up and execution is high. Furthermore, testing of a complete construction in fire is a formidable task as evidenced by the full-scale fire tests undertaken on a building by the Building Research Establishment in the great airship hangar at Cardington, England.

2.5.2 Tabulated data

Current building fire engineering practice is largely based on the application of codes of tabulated data whereby the engineer designs in accordance with pre-determined requirements based on generic occupancies or classes of fire risk. These, largely prescriptive codes, are essentially rigid and restrictive and do not allow for engineering thinking. Although the cheapest to implement, they are the least accurate of the three analytical methods. The safety level achieved by this method can vary significantly. In many cases it is very conservative and not cost efficient [AND 98]. The fire resistance requirement in the tabulated data is expressed by target fire resistance ratings for members exposed to the standard ISO 834 fire. This means that a structural member should be designed in such a way that it does not collapse within 30, 60, 90 or maybe 120 minutes. No classes in-between are available. Hence, if a test specimen collapses after 59 minutes it will be categorised in the R30 fire class, irrespective of the loading level. This also applies to standard tests. Such deficiencies have provided the driving force for the development and wider acceptance of performance based methods.

Engineers apply prescriptive methods for the fire resistance of building elements or sub-assemblies from tabulated data presented in the codes and standards. Eurocode 2 [EC2 05a] gives tables specifying minimum dimensions including axis distance (in mm) for fire resistances ranging from 1/2 hour to 4 hours for concrete beams, columns and floors. The data also distinguishes between simply supported and continuous constructions for flexural members for both reinforced and prestressed concrete. The tables are based on the assumption that the elements are supporting the full design load. The tabulated data for simply supported elements are based on the steel reinforcement retaining a proportion of its strength at high temperatures; reinforcing bars and prestressing tendons are considered to retain about 50% of their ambient strength at 550°C and 350°C respectively. If any dimension of a particular construction is less than the minimum specified in the tables and it is not possible or desirable to increase it to meet the requirements, the fire resistance may be enhanced by the application of a protective coating, system or membrane [MOR 88]. It is necessary to appreciate that these periods do not signify the duration of an actual fire. For example a 60 minute fire does not imply that a construction is expected to withstand a fire of 60 minutes duration but will withstand a fire of a longer or a shorter duration whose *severity* corresponds to the 60 minute furnace test.

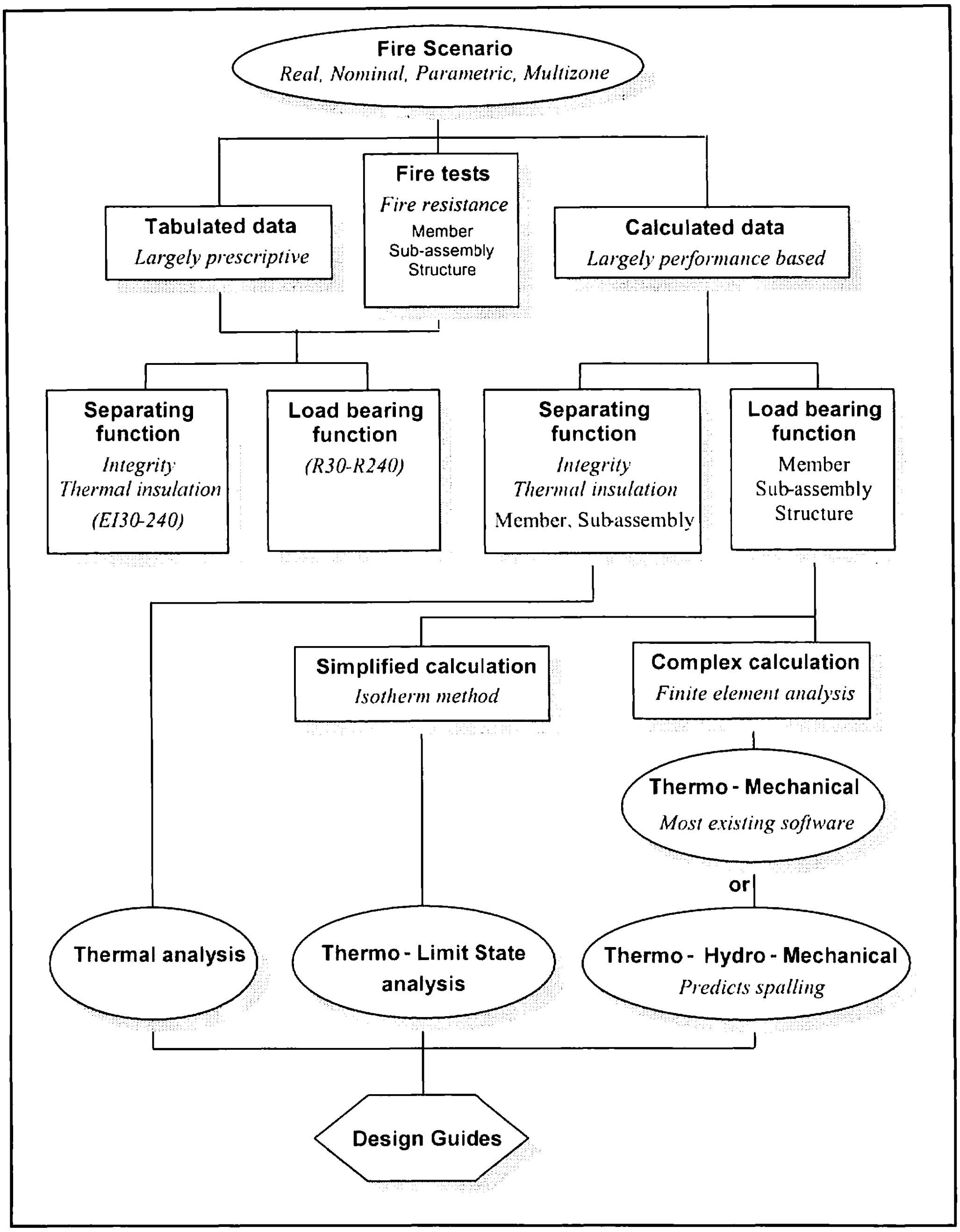


Figure 2.4 Three main options of concrete fire assessment and design process [KHO 00].
Note: Calculation based methods are becoming acceptable in an increasing number of countries.

Specific provisions of test for fire resistance of elements of structure in terms of the three performance criteria of collapse, integrity and insulation are given in the ISO standards. The regulations also set out the minimum periods of fire resistance in minutes for elements of structure at basement, ground, and upper levels of various types of building. The regulations state that where one element of structure supports or carries or gives stability to another, the fire resistance of the supporting element should be no less than the minimum period of fire resistance for the other element (whether that other element is load-bearing or not). Circumstances for varying this principle are also given in national codes.

Tabulated data on minimum axis distances for beams and slabs are available in Eurocodes 2 only for ISO 834 fire but not for parametric fires. However, the Swedish manual on fire engineering design of concrete structures [AND 92a-b] provides such data for parametric fires as well.

2.5.3 Calculation based methods

Fire engineering calculation provides a cost-effective and flexible method of assessment normally superior to prescriptive methods. A given problem can be studied for different fire scenarios, geometries, material properties, loading or support conditions. This can be performed in a relatively short period of time thus allowing a better understanding of the behaviour of the structure subjected to fire until collapse. Moreover, computer programs could even simulate structural conditions that are very difficult to study in a fire test. In the performance-based design, the structure is not allowed to collapse during the complete fire process - including the cooling phase [AND 98].

Fire engineering calculation methods can be classified into three categories of increasing sophistication and complexity. These are (Figure 2.4):

- Simplified calculations based on limit state analysis.
- Thermo-mechanical finite element analysis
- Comprehensive thermo-hydro-mechanical finite element analysis

2.5.3.1 Thermal analysis

When fire resistance is assessed by calculation, the first step would be to determine the thermal response in the form of time-dependent temperature distribution in the concrete structure. In integrated models the temperature distribution would be calculated along with the hydral and mechanical states for each time step. The thermal response provides the final answer only for the thermal insulation criterion but not for the load-bearing capacity or the complete answer for separating function (integrity). However, thermal analysis provides a necessary link to the structural analysis (Figure 2.4).

Thermal analysis can range from the simple scenario of a uniform specified gas temperature at the boundary to the varied thermal boundary condition based on heat flows or temperature. Most simply, the temperature distribution can be determined with the aid of default graphs of concrete cross sections with isotherms for structures exposed from 1 up to 4 sides at default times intervals. These graphs have been developed for standard fire exposure as well as for parametric fires [AND 92b]. For members with a constant cross sectional geometry, the temperature distribution over one cross-section is considered. Since a uniform gas temperature throughout the compartment is assumed, the temperature profile in the structure along the member's axis will also be constant. The effect of thermal shielding by other structural parts near the supports is generally neglected.

Knowledge of the development of temperature distribution in concrete structures is the first key step in the understanding of the structure's behaviour in fire. Air temperatures in fires can exceed 900°C. However, the good insulating properties of concrete mean that the temperature gradient is large and only the temperature of the outside layer is markedly increased, while the temperature of the internal concrete remains comparatively low.

2.5.3.2 Simplified limit state analysis

Having determined the temperature distribution for the structural element either from the published literature on a similar element or from thermal analysis, a simplified limit state analysis is carried out using a technique first proposed by Anderberg [AND 76] and now introduced into Annex B the Eurocode 2 [EC2 05c]. He suggests a very simple method of analysis based on the hypothesis that the thickness of the damaged siliceous concrete is assumed to equal the average depth of the 500°C isotherm in the compression zone of the cross-section (Figure 2.5). However, this isotherm was selected following tests on siliceous concrete. The Eurocode reference to 500°C as being the dividing line between damaged and undamaged concrete cannot be applied to concretes containing other aggregates which have lower thermal stability than siliceous aggregates. These, for instance, include flint (250°C) and Thames River gravel (350°C). For such concretes, the isotherm temperature may well be below 500°C, or even below 400°C. Therefore, the choice of the temperature 500°C isotherm in the Eurocodes would be misleading because, the isotherm temperature will depend upon the type of concrete used and its characteristic strength loss against temperature. Damaged concrete (i.e. concrete with temperatures in excess of the "critical" temperature) is not expected to contribute to the load-carrying capacity of the member, whilst the residual concrete cross-section is assumed to retain its full initial values of strength and modulus of elasticity. Anderberg suggests that the "isotherm method", or "Effective cross-section method" is applicable to a reinforced concrete section with respect to axial load, bending moment and their combinations. For this method to apply, there should be minimum dimensions of the member depending upon fire resistance time or fire load density [EC2 05c]. Another more detailed method called the "Zone Method" was proposed by Hertz and has also been introduced in Annex B the Eurocode 2 [EC2 05c].

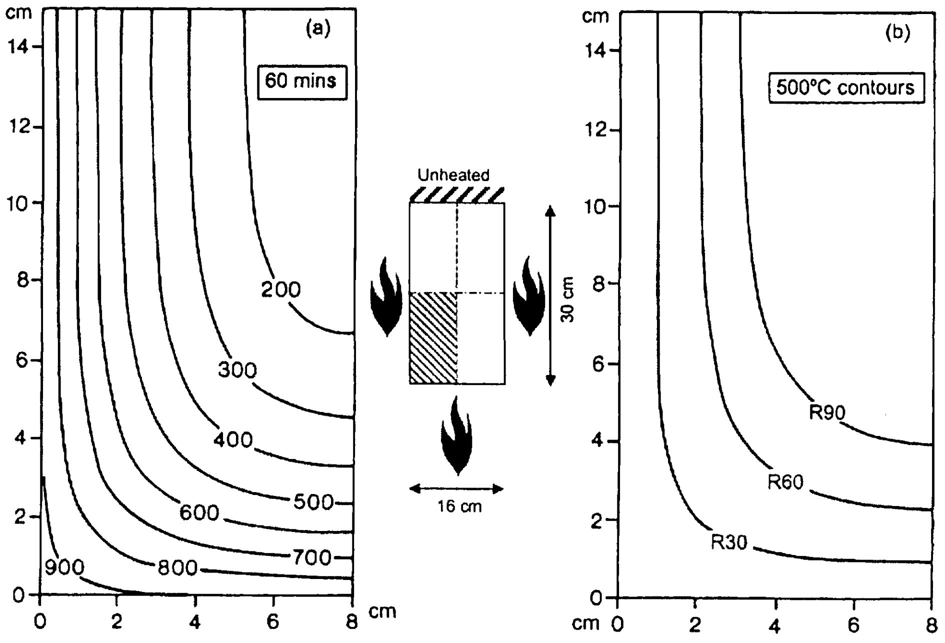


Figure 2.5: Temperature contours in a quarter section of a concrete beam heated on three sides in ISO834 (or BS476) fire: (a) Contours after 60 minutes; (b) 500°C isotherms after 30, 60 and 90 minutes [EC2 05b] valid for siliceous concrete but not for all concretes.

2.5.3.3 Thermo-mechanical finite element analysis

The bulk of performance based fire dedicated software dating back to the 1970s falls into this category. The thermal and mechanical analysis are normally interfaced and not integrated. In other words, the thermal calculation is carried out first for the entire duration of the fire and then fed into the mechanical analysis program to produce the stresses and strains for the member/structure - there being no interaction between the two analyses - and moisture effects are also absent. Nevertheless, validation of the results from such programs has consistently shown that the deformations of simple elements can be reasonably accurately predicted by such methods providing the load induced thermal strain (LITS [KHO-06a]) is incorporated into the model, particularly for columns. LITS is dealt with in chapter 5 of this Bulletin. The modelling of LITS is relatively simple in fire dedicated programs where the duration of the analysis is in minutes or a few hours. This is because LITS is largely a function of temperature rather than time. Therefore, time-dependent creep functions can be dispensed with, without prejudicing the outcome. The absence of a moisture migration analysis means that the evaporation plateau and explosive spalling cannot be predicted. Depending on the finite element types and constitutive models used, loss of load bearing capacity due to the propagation of thermal cracks can be calculated [FEL 04]

2.5.3.4 Comprehensive thermo-hydro-mechanical finite element analysis

A comprehensive analysis (Figure 2.6) would incorporate thermal, hydral and mechanical analyses in a fully integrated and interactive model capable of predicting explosive spalling (e.g. in tunnels) or the moisture state of the concrete containment of nuclear reactors (important function as a biological radiation shield). In most practical cases, however, simpler methods are adequate for the analysis of structures in fire.

The first such model was developed in the 1970s by Bazant and Thongutahi [BAZ 79], albeit with limitations in the modelling. For example, the hydral component is considered to be a single phase smeared fluid and spalling is not incorporated.

A more advanced model was developed in 1999 at Padua University in co-operation with ENEA in Rome and Imperial College in London as part of a multi-national programme of research funded by the European Commission [KHO 02a] [SCH 02]. Called HITECOSP (High Temperature Concrete Spalling) it is a fully coupled non-linear model designed to predict the behaviour, and potential for spalling, of heated concrete structures for fire and nuclear reactor applications. Concrete is considered as a multiphase material consisting of a solid phase, two gas phases and three water phases. Refinements (such as the effect of damage on permeability and non-linearities due to temperature) are included and chemical-physical phase changes are incorporated such as hydration-dehydration, evaporation-condensation, and adsorption-desorption. Phase changes in concrete are, for the first time, incorporated directly in the transport mechanism of any concrete model. Melting and evaporation of PP fibres are modelled as phase changes.

The complete behaviour of concrete in the elastic, inelastic and plastic ranges up to fracture is described. Previous models were based on elastic fracture only. The physical model is described with emphasis being placed upon the real processes occurring in concrete during heating based on tests carried out in several major laboratories around Europe as part of the wider HITECO research programme. A number of experimental and modelling advances are presented in this work. The stress-strain behaviour of concrete in direct tension - determined experimentally for the first time - is input into the model. The hitherto unknown microstructural, hygral and mechanical behaviour of high strength concrete and ultra high strength concrete were determined experimentally and the information is also built into the model. It is also the first time that such a complex model has been developed with the aim of predicting the behaviour of concrete at high temperatures in general and explosive spalling in particular. The fluid phase is considered to consist of water, dry air and vapour. The water phase consists of free capillary and physically bound water. Chemically bound water is considered part of the solid skeleton until it is released on heating. The solid is fully deformable and able to experience elastic, damage, thermal, creep, shrinkage, plastic, cracking strain processes. Both "basic" and importantly "transient" creep phenomena are included. Outputs are presented in the form of three-dimensional colour diagrams of temperature, vapour pressure, water saturation, and damage distributions. An example output for a concrete column subjected to fire is shown in (Figure 2.7).

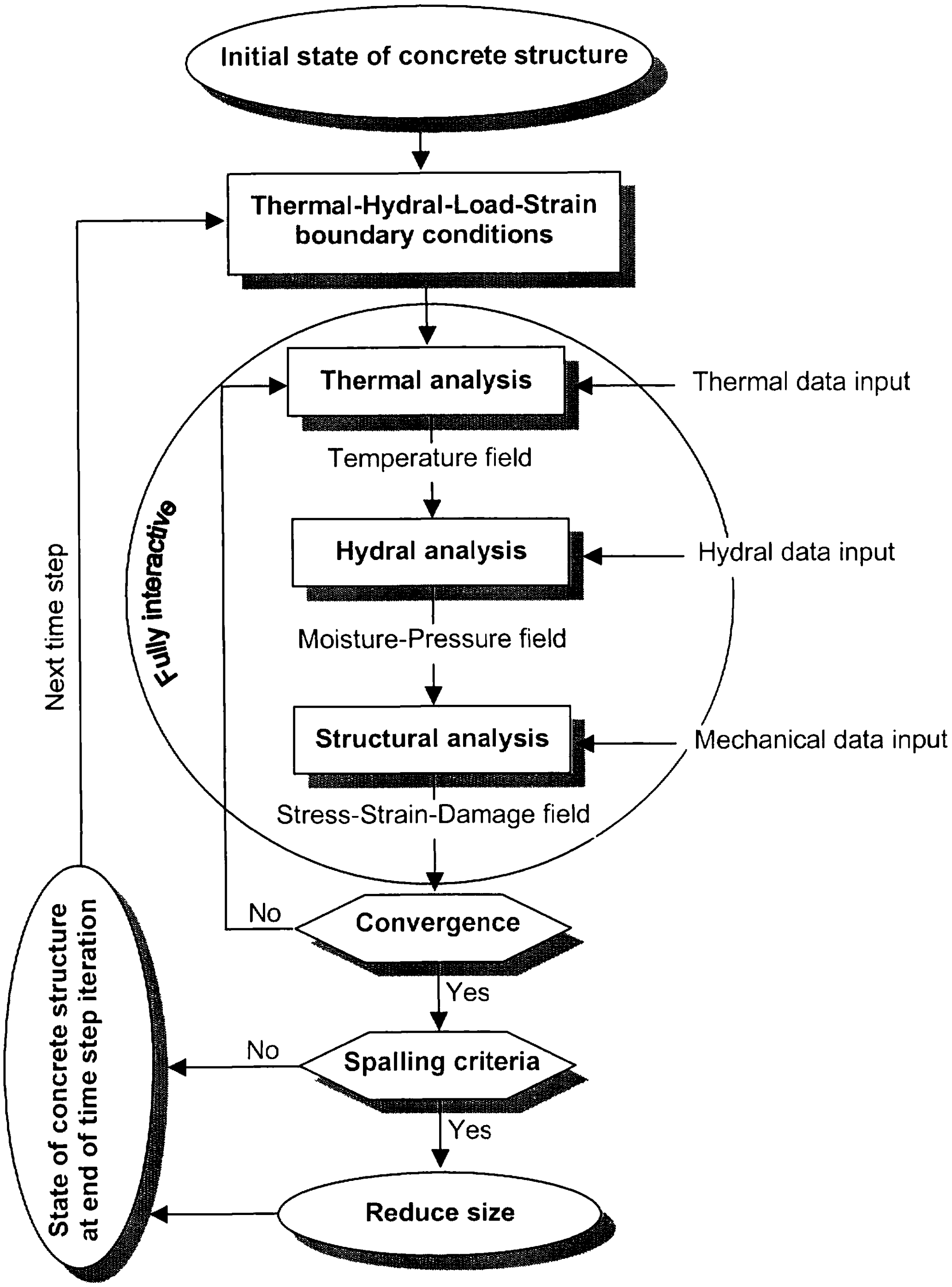


Figure 2.6: Simplified flow chart of thermo-hydro-mechanical finite element analysis of heated concrete structures including spalling [KHO 00].
Note: Most fire dedicated finite element software are thermo-mechanical and exclude the hydral component. The thermal and mechanical analyses are usually interfaced and not integrated, hence the inputs and outputs are shown separately. With a fully integrated and interactive model, the input data are not separated because they all contribute to the overall analysis.

2.5.3.5 Details of the advanced calculation method

The aim of the procedure is to analyse the behaviour of concrete when treated as partially saturated porous material under the action of high temperatures and to apply the results to the structural field. At the proposed level of analysis, the phenomena connected to the chemo-thermo-hygrometry and mechanics of materials are considered in a simultaneous fully coupled fashion, referring to one of the most advanced models of this type actually available in the technical-scientific literature. The investigation proposed at this level allows the identification and study of the complex interactions between solid, liquid and gaseous phases inside concrete when exposed to high temperatures, so to exploit a space-time mapping of the main and secondary variables governing transport phenomena. Many applications are feasible in structural field. They pertain to a wide class of structures in the civil and industrial construction field, such as buildings, tunnels and nuclear reactors.

In general mass transport in concrete is inclusive of dry air flows due to free or forced convection or to infiltration through material fractures and pores, as well as of vapour transport due to diffusion, liquid flows due to diffusion, to capillary suction or gravity and further flows associated with phase changes due to condensation / evaporation, freezing / thawing, ablimation / sublimation, adsorption / desorption and to further phase changes of a chemical nature. For concrete the carbonation / decarbonation and the transformations of quartz, allow to the analysis of almost all aspects of concrete behaviour together with physical phase changes. The moving fluids through porous medium give a crucial contribution to the heat transport. The evolution of thermal and pressure fields produced by the thermal load applied, as well as by mechanical loads, are responsible for free deformations in the material, which are strictly connected to the above phase changes. In particular, deformations due to the described chemical-physical phenomena strongly contribute to explain the so called creep. Free deformations are transformed into stresses, because of geometrical configurations, applied boundary conditions to the specific examined structure, and of the constitutive relationships of concrete which is of a coupled elastoplastic-damage type. For this purpose the adopted constitutive theories are able to represent adequately the material behaviour in different uni-axial, bi-axial and tri-axial working regimes. When high temperatures develop, adaptation, extension and information on material data are needed to maintain thermodynamic consistency of the model.

The material properties pertaining to heat and mass transfer are basically anisotropic but can be reduced to orthotropic in the case of fractured concrete (e.g. thermal conductivity and capacity, vapour and liquid water permeabilities). Similarly mechanical characteristics are different with directions (e.g. compressive, tensile and shear strengths, thermal creep, thermal dilatation of aggregates in damaged/fractured concrete). The evolution of the above properties is significant during load history as a result of the changes in the microstructure of pores (porosity, distribution of pores, microfracture). The material properties depend also on the amount water in the pores, especially in the initial state, and by the successive aging and creep phenomena. The external actions have a critical effect on the spalling behaviour of concrete: an external load or pretensioning increase considerably the risk of spalling in concrete.

In this method of assessment of fire resistance, a physical-mathematical model is set up, then translated in a numerical fashion, to study chemo-thermo-hygro-mechanical phenomena of concrete, starting from an existing general model [KHO 02a] [SCH 02]. The base equations, i.e. mass conservations, energy and momentum conservation, are obtained through the use of Volume Averaging techniques. A fully coupled approach is used to describe the behaviour of the multiphase partially saturated porous material where all the most significant phase changes are taken into account. Moreover, the dependence of material properties on temperature and pressure, damage, carbonation (if any) and all thermal, hygral and mechanical couplings are included. Particular care must be taken with respect to thermal exchanges at boundaries and, where needed, to the thermal-fluid-dynamics of the surrounding environment of the analysed structural elements.

The final aim of the assessment process is to explain the behaviour of concrete under the action of fire and evaluate the risk of explosive spalling phenomena, so as to apply the results of material research to structural macro-elements currently used (beams, columns, plates and 3D elements). The recent fires which occurred in various road tunnels have shown the importance and need for this approach, although the problem of fire in buildings is a risk exhibited by virtually all building constructions, as a consequence of accidental events like explosions of gas pipelines, or intentional events. The above aspect is even more important if related to buildings having large gatherings of the public and buildings were working activities are carried out. The need to understand how concrete behaves in its inner structure is connected to the need to design using the most appropriate measures to avoid or at least mitigate the negative effects of high temperature, so to guarantee adequate safety levels to the structural elements that are manufactured using the above material.

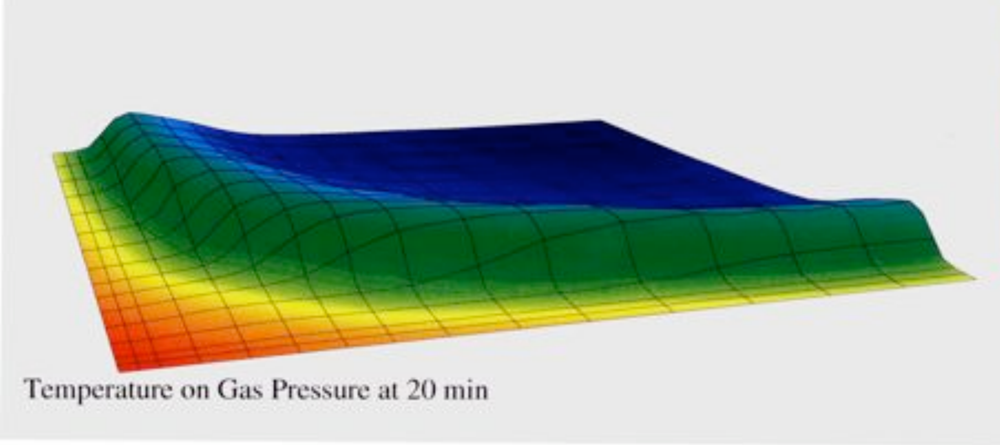


Figure 2.7: Axonometric profiles of temperature (shown by colour) and gas pressure (shown by undulations) in a C60 HSC concrete column 20 minutes into a fire (radiative heating only) [PES 00] [KHO 02a].

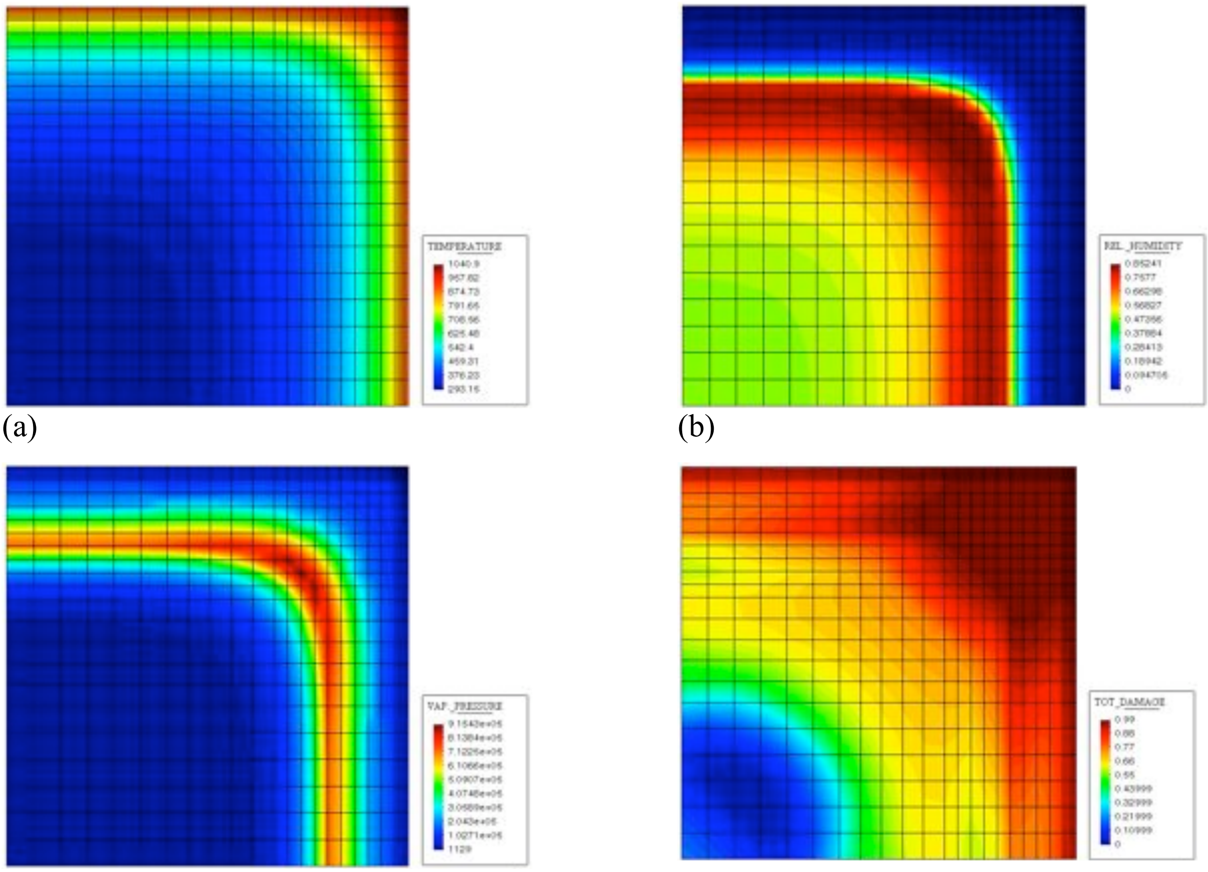
In the analysis of chemo-hygro-thermal and mechanical behaviour of concrete at high temperatures, various physical-chemical coupled phenomena must be taken into account. A fundamental role in the chemical processes and in the heat and mass transfer is played by dehydration. Moreover, in concrete, at temperatures higher than the critical point of water of 374.15^{o}C when water cannot exist in a liquid state, primary chemical reactions occur as calcium carbonate decomposition and silica components transformations.

In concrete at first heating, the application of non-uniform temperatures induces an increase of vapour pressure and is responsible for an initial vapour migration towards the lower temperatures zones. Initially the pores are not physically saturated. In such a material the typical percentage of not saturated volume is equal to 2-4% for the case of plain concrete. These partially saturated pores play a fundamental role in the initial migration of water since they act as humidity storage, avoiding an excessive increasing of the pressure during heating. After the early moisture migration, these initially unfilled pores are filled and a “saturation plug” (or “moisture clog”) forms and then water moves under the action of a hydraulic gradient governed by saturation vapour pressures at the two ends of the saturated zone. Under steady-state conditions the saturation zone moves away from the hottest region and increases in length simultaneously as water accumulates at the upstream face. The phenomenon of moisture clog may be responsible for explosive spalling of concrete, because of the high values of water vapour and liquid pressure reached during the heating process.

It is possible only at a general level to standardize the behaviour of concretes within one model of partially saturated porous medium subject to high temperatures. The material nature, together with homogeneity/non-homogeneity, isotropic/anisotropic properties play a fundamental role in the final behaviour of the model chosen to carry on the type of analysis proposed here. At higher temperatures it is the cement matrix which becomes the weak point of the material, showing low mechanical strength. With increasing temperature the aggregates progressively expand as long as they are not chemically altered, while the cement matrix, after an initial expansion, is subject (over 150°C) to a progressive shrinkage. These two opposite phenomena involve a micro-cracking process which explains the subsequent damaging of the material microstructure. Further, low permeability inhibits water mass transfer causing high gas pressure values, crack-opening and a further increase in intrinsic permeability. An important role is played during fire by the large load induced thermal strain (LITS) [KHO-06]. This strain plays an important role in the relaxation of thermal stersses.

2.5.3.6 Structure of the model.

The model to analyse chemo-thermo-hygro-mechanical behaviour of concrete is based on four balance equations: i) mass of dry air, ii) mass of the water species (liquid + vapour, taking phase changes into account), iii) enthalpy of the whole medium (including latent heat of phase changes and heat effects of hydration or dehydration processes) and iv) linear momentum of the multiphase system. Inclusion in the model of the carbonation process needs other balance equations, such as mass conservation equation of calcium carbonate and mass conservation equation of carbon dioxide, which are the new two components of the "concrete" system. Changes in the material properties caused by temperature and pressure changes, damage, plasticity together with coupling between thermal, hygral and mechanical phenomena are taken into account. The model further allows the incorporation of sorption/desorption isotherms hysteresis and creep (e.g. LITS) behaviour.



(a) (b)

(c) (d)

Figure 2.8: Example of a quarter HSC column section distribution of (a) temperature, (b) relative humidity, (c) vapour pressure, (d) damage exposed when to a mix of radiative and convective heating conditions[PES 00] [KHO 02a] See also related Tables 2.2 and 2.3.

The multiphase model of concrete is completed by an appropriate set of constitutive and state equations, as well as basic thermodynamic relationships. The governing equations of the model are expressed in terms of the chosen state variables: gas pressure pg, capillary pressure pc, temperature T, displacement vector of the solid matrix u and, if carbonation is included, carbon dioxide pressure and calcium carbonate density. Due to the curvature of the interface between the capillary or bound water and the gas phase inside the pores of the porous medium considered as a capillary porous body, the equilibrium water vapour pressure pgw differs from the saturation pressure pgws and can be obtained from Kelvin's equation. For fluid phases the multiphase Darcy's law is applied as the constitutive equation, while for gaseous constituents, i.e. dry air, water vapour and moist air, the equation of state of perfect gases and Dalton's law are assumed.

Enhancements concerning the possibility to treat the gas phases in concrete as real gas in the range of temperature of 160°C - 375.15°C, is introduced in the model and in the code. As far as the constitutive equations describing the mechanical behaviour of concrete are concerned, a modified form of Bishop's stress responsible for all deformations in concrete is incorporated. Then the constitutive relationship for the solid skeleton is assumed in rate form to include thermal, autogeneous and creep effects.

Material data of concrete at high temperatures complete the set of equations needed. Kinetics and amplitude of the processes described above are controlled by the material properties, i.e. governing and constitutive equations will contain several coefficients describing material non-linear behaviour. The material properties include heat and mass transfer properties (heat conductivity and heat capacity, permeabilities to vapour and liquid water) as well as mechanical properties (strengths, thermal creep, thermal dilatation of the aggregates if it is the case, etc.). These properties evolve significantly under applied loading as a result of changes in the microstructure of pores (porosity, pore size distribution, microcracking). The material properties also depend on moisture content of pores. For example, the saturation degree with liquid water S (considering together hygroscopic and capillary water), is an experimentally determined function of capillary pressure and temperature (hysteresis phenomenon is included using neural networks). This relationship, usually presented in the form of sorption isotherms, is fundamental for realistic modelling of hygro-thermic behaviour of a specific concrete. It should thus be determined during sorption tests at several temperatures. These are difficult to determine experimentally. Thermal dilatation of the aggregates plays an important role in damage and varies significantly from one aggregate to the other. For example calcareous aggregates dilate less than siliceous ones.

Calibration and validation of the model is then needed to obtain useful software, available at an industrial level. For this purpose the concrete model is calibrated using well performed experimental results. The finite element codes produced, supplied by graphical pre- and post-processing routines for an easy treatment of data and results, incorporating all the above described features. Numerical tests at different scales should be carried out for each application. At lower scales, cylindrical samples can be considered, while at larger scales, the influence of size and shape of the element governing temperature and moisture fields must be checked, as well as the whole chemo-thermo-hydro-mechanical history. Similarly, the external action effects, in particular the mechanical and thermal loading, which may have a critical effect on spalling behaviour of concrete, are incorporated. The thermal load is of course a determining factor; in most cases, the risk of spalling increases with the heating rate. However, there may be many exceptions to this: for example, a higher heating rate in a column may generate some macrocracks which would help to transfer water from its core without its strength reducing too much. At lower heating rates it would be impossible to transfer all water from the core and explosive spalling would occur. Just for simulating the above situation, numerical examples dealing with fast and slowly heated cylindrical specimens as well as walls, beams and columns, must be properly investigated.

2.5.3.7 Example of analysis

This example [PES 00] deals with a square column (30×30 cm) made of the C-60 HSC subjected to the standard fire. Considering the shape, a quarter of the column is discretised by use of 625 (25 x 25) eight-noded serendipity elements. The first 20 minutes of the hygro-thermal and mechanical performance of the column in fire is simulated using a time step length equal to 1s.

Parameter	**Symbol**	**Unit**	**Value**
Porosity	φ	[-]	0.082
Intrinsic permeability	K	[m^2]	2×10^{-18}
Apparent density	ρ	[kg/m^3]	2564
Specific heat	C_p^s	[J/kgK]	855
Thermal conductivity	λ	[W/mK]	1.92
Young's modulus	E	[GPa]	34.52
Poisson's ratio	ν	[-]	0.18
Compressive strength	f_c	[MPa]	60
Tensile strength	f_t	[MPa]	6.0

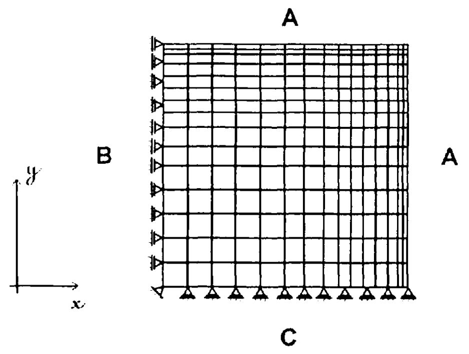
Table 2.2: Characteristic properties of the C-60 concrete at ambient temperature [PES 00].

The column is assumed to be initially in hygro-thermal equilibrium with the surrounding air having relative humidity of 60%, gas pressure corresponding to atmospheric pressure, i.e. 101325 Pa, and temperature of 25°C. The main characteristic material parameters are given in Table 2.2 and the boundary conditions employed in simulations are shown in Table 2.3.

During heating of the column we observe a gradual increase of temperature in the surface zone, Figure 2.8 (a), where moisture content and relative humidity decrease rapidly, Figure 2.8 (b), reaching very low values. Sharp fronts with very high gradients of temperature and relative humidity, moving towards the central part of the sample, are visible, Figure 2.8 (a), (b). In this zone, rapid evaporation causes an increase of vapour pressure up to values exceeding 0.9 MPa, Figure 2.8 (c). This phenomenon, together with thermal dilatation of the outer part of the sample, while the internal remains still at the initial temperature, causes high tensile stresses and resulting material damage Figure 2.8 (d). This effect is particularly important in the corner zone of the sample, which is typical for concrete elements of a similar shape. At the same time the temperature increase causes significant degradation of concrete in the heated zone, additionally increasing deterioration of the surface zone. High values of the vapour pressure and the total damage parameter in the corner zone, close to the surface, indicate a serious risk of explosive spalling.

Numerous experimental tests showed that for this kind of concrete element geometry spalling often occurs in the corner zone, what is qualitatively confirmed by the results of this numerical simulations.

In the inner zone of the column, higher values of the relative humidity are observed as a result of advective flow of vapour caused by vapour pressure gradient, Figure 2.8 (b), (c), and consequent vapour condensation in the colder, internal part of the element. Hygral and thermal phenomena, as well as concrete deterioration have a very complex and strongly coupled character, thus they cannot be analysed with sufficient accuracy by means of uncoupled or simplified models.



Side	Variables	Values and coefficients
A	u_x	$u_x = 0$
	u_y	$u_y = 0$
	p^g	$p^g = 101325$ [Pa]
	p^c	$p^{gw} = 1000$ [Pa], $\beta_c = 0.18$ [m s^{-1}]
	T: convective	T= ISO-Fire [K], $\alpha_c = 18$ [W m^{-2}K^{-1}]
	T: radiative	$e\sigma_0 = 5.1 \cdot 10^{-8}$ [W m^{-2}K^{-4}]
B	u_x	$u_x = 0$
	p^g	$q^g = 0$
	p^c	$q^{gw} = q^w = 0$
	T	$q^T = 0$
C	u_y	$u_y = 0$
	p^g	$q^g = 0$
	p^c	$q^{gw} = q^w = 0$
	T	$q^T = 0$

Table 2.3: Heat and mass coefficients and Boundary conditions for a column exposed to radiative and convective heating [PES 00] (see also related Table 2.2 and Figure 2.8)

2.6 Failure modes in fire

All relevant failure modes should be considered when evaluating the fire resistance of a concrete structure with respect to the load bearing capacity. Depending upon the loading arrangement, the heating history and the type of structure, the following failure modes could occur:

- Bending failure
- Buckling/compression failure
- Anchorage/bond failure
- Shear or torsional failure
- Spalling failure

2.6.1 Bending failure

Bending failure is by far the most common type of failure for horizontal members such as beams and one-way slabs and not so much in two-way slabs. A design assumption (not actual behaviour) is that bending moments cause tensile stresses in the reinforcement and compressive stresses in the concrete. Sagging bending moments cause tensile stresses in the bottom reinforcement and compressive stresses in the concrete at the top, hogging bending moment creates tensile stresses in the top reinforcement and to compressive stresses in the concrete at the bottom. Given the fact that only fire exposure of the bottom side is considered, failure can occur by either rupture of the reinforcement which is common for sagging bending moments or crushing of the concrete in the compression zone, which is more likely for hogging bending moments near the supports. Near the supports in continuous beams or slabs, also rupture of the hogging reinforcement can occur, caused by large rotations at the support rather than by a decrease of the mechanical properties of the hogging reinforcement. Tests have demonstrated that failure in two-way slabs supported on their four edges involves more membrane tension than bending.

2.6.2 Buckling/compression failure

Buckling failure is the most relevant failure type for vertical members such as columns and walls. Due to the heating of the structure, the Young's modulus of both concrete and reinforcement reduces, which leads to a reduced buckling strength. Moreover, the strength of concrete and steel reduces at elevated temperatures, thus leading to a reduced bending moment capacity. The combination of the reduced strength and stiffness leads eventually to buckling failure. Reinforced concrete members under compression (e.g. columns) usually fail in fire due to concrete failure in the compression zone as its strength diminishes with heating. The loss in strength of the reinforcement with heating is of minor importance in cases where the steel content is small, but not at high steel contents where both the concrete and steel influence the load bearing behaviour. For compression members heated on one side, typically for walls, failure can occur by compression in the concrete or by tension in steel, depending on the direction of deflections at failure, either toward the fire compartment (typically in slender walls) or toward the other direction (typically in shorter walls).

2.6.3 Anchorage/bond failure

Anchorage failure can occur if the stresses in the reinforcement cannot be developed over the available embedment length. The reinforcement will be pulled out. In concrete structures with mild steel and without prestressing, anchorage failure is very rare. Moreover, in these structures, mechanical anchorage can be provided by bending of the reinforcement at the ends. However, in prestressed structures using pre-tensioned reinforcement, anchorage failure can govern the behaviour. In these structures normally strands are used as reinforcement. The bond stresses between the reinforcement and the embedded concrete are affected by the decrease of the strength and stiffness properties of steel and concrete. Moreover, the non linear temperature distribution causes thermal stresses in the reinforcement that lead to additional slip of the strands. Failure of reinforced concrete members may occur in fire when heating reduces the bond strength between the steel and concrete. Recently, an advanced FE model was developed [FEL 04] that can describe anchorage failure of prestressed concrete using pre-tensioned strands, taking into account the splitting cracks and additional slip caused by the thermal deformations.

2.6.4 Shear or torsional failure

Shear and torsional failure do not often dominate the behaviour of fire exposed concrete structures. Shear stresses are mainly transferred near the centre of the cross-section, which is relative cold compared to, for instance, the reinforcement required to take bending moments. Nevertheless, shear and torsional failure can occur, especially in concrete structures without shear reinforcement. During fire, if deformation is resisted, the hot fire exposed side of the cross section usually expands (depending mainly upon the aggregate type) and is induced into compression, while the cold unexposed side is induced into tension which, therefore, may develop cracks. Consequently, the shear stresses are transferred through the section on the compressed and hot exposed side. Moreover, the decrease of the mechanical properties of steel and concrete reduce the shear capacity. Shear or torsion failure in fire is influenced by the concrete tensile strength and is much more complicated to determine than bending failure, because of limited experimental experience and the more brittle behaviour requiring the consideration of thermal deformations. As torsion members with a defined fire resistance are rare, the solution of the problem may be shifted to individual cases if any. There is, however, practical interest in shear beams which are usually designed with a special shear reinforcement. Attention should be drawn to the more likely shear failure in prestressed beams with an I section, because the web heats up faster than the flanges

2.6.5 Spalling failure

Spalling is discussed in chapter 6 of this Bulletin. It is actually not a strauctural failure mechanism. Spalling may lead to rapid loss of cross-sectional dimensions, which may invoke a "real" failure mechanism as buckling, bending failure, etc.

3 Fire and concrete material

Strictly, fire "resistance" is a concept applicable to elements of the building structure and not a material, but the properties of a material affect the performance of the element of which it forms a part. Fire generates heat which influences the microstructural, thermal, hydral and mechanical behaviour of concrete. Understanding of the science and behaviour of heated concrete as a material is not only intended to help explain how existing concrete is likely to behave in fire, but also importantly how concrete material can be designed and optimised for better fire performance in future applications.

3.1 Complex behaviour has lead to versatility

It should be noted, in general, that concrete, during first heating especially is a highly unstable and complex material. Changes in the microstructure occur as temperature rises. The thermal-hydral-mechanical properties also change with time and temperature depending upon the heating rate, initial moisture condition, boundary conditions, geometry and size of the heated member, loading condition, type of constituents, chemico-physico interactions etc. Being probably the most complex of building materials and one of the most abundant, makes this a subject of great practical importance. Therefore, great attention should be paid by the researcher who produces the data in unravelling and understanding the complexities in order to present the subject to the practising engineer in whatever form (e.g. texts, diagrams, tables, numerical modelling output) that are readily usable, as close to reality as possible, and as reliable as possible. This requires good science and even good art.

3.2 Causes for misunderstanding

The guidance given in this section naturally depends upon the outcome of research and experimentation from different institutions, different countries, and from across the generations. The complexity of this subject has resulted in wide scatter in the data, even a single property, from different sources. Some of the key reasons emanate from the different use of, for example:

- different concrete types labelled generally as "concrete".
- different environmental conditions and testing conditions (curing, load-heat sequence, moisture condition, loading, boundary conditions etc.)
- different test equipment and test methods, even the wrong test methods on occasions.

Coupled with sometimes poor understanding of the subject, it is not surprising that research data disagree and even conflict.

3.3 Key influences

3.3.1 Concrete type

The testing and use of different concrete types has led to the most confusing and misleading of issues. Many researchers in the past have tested "siliceous" concrete and simply labelled it as "concrete". The data were entered into the standards and codes without proper qualification. The testing of different types of concrete materials in fire results in a huge difference in behaviour. For example, thermal strain depends upon the aggregate used, which in turn influences the thermal and parasitic stresses within the concrete during heating. Moisture migration and loss depends largely upon the permeability of the cement paste (and that of the aggregate if it contains continuous porous). Here again large differences in behaviour can be found between concretes with low or high permeability in terms of pore pressures and moisture migration, thus influencing materials properties and the likelihood of explosive spalling occurring. The thermal stability of the concrete depends largely on the thermal stability of the aggregate, a key point usually not considered even in recent major tests. Pre-heated concrete behaves differently from concrete heated for the first time. Concrete heated in the "sealed" condition behaves radically differently from concrete heated "unsealed" from which moisture is allowed to escape. The foregoing is by no means exhaustive and only presented for caution and guidance.

3.3.2 Test regime

This is a subject area that has also resulted in much confusion. Most researchers start their work unaware of the key role played by the testing regime, in terms of loading-heating-time sequence, let alone boundary conditions, initial moisture content, type of concrete, geometry and size of specimen. A global diagram (Figure 3.1) has been constructed as guidance to present an overview of thermo-hydro-mechanical properties of concrete dependant upon the load-heat-time sequence. The "test regime" comprises a given condition or a combination thereof. Not shown in Figure 3.1, however, is the influence of moisture boundary conditions.

The thermal mode may consist testing a concrete specimen:

1. "cold" prior to heating;
2. "transient" during heating;
3. "steady state" elevated temperature;
4. "transient" during cooling;
5. "residual" after cooling to ambient temperature;
6. "post cooling" i.e. a period after cooling,
7. "second heating" and so on.

Essentially tests 1, 3, 5 and 6 are carried out at constant temperature, whereas tests 2, 4 and 7 are performed under transient temperature regimes.

While tests could be carried out under varying temperature regimes, the usual practice for clarity and simplification is to carry out tests either at constant temperature (e.g. 300°C) or at constant rates of heating (e.g. at 2°C/minutes). As for cooling, it could be "natural" or "forced".

Heating rates of, for example 2°C/minute for mechanical properties, while too slow to simulate fire conditions are employed by scientists - and recommended by the RILEM Committee [RIL 07, 08] – in order to separate as far as possible the "material" from the "structural" effect of heating a specimen the size of, say 6 cm in diameter and 18 cm in length [KHO 84]. In this respect fast heating will introduce thermal and moisture gradients and hence thermal stresses within the specimen. Smaller size specimens (and in powder form) are generally exposed to higher heating rates because the size limits the "structural" effects. The limitation in heating rate, for some materials properties, have been shown to provide more accurate indication of the "material" behaviour. For example, the load induced thermal strain (LITS) in compression which includes a transient creep/strain component, is for practical purposes independent of time exposed in the unsealed condition [KHO 85b]. LITS data were similar for small specimens of 6cm and 3 cm diameters heated at 1°C/minute [KHO 87]. However, in tension, even in specimens as small as 6 cm diameter and heated as slow as 0.5°C/minute, thermal stresses develop with significant influence on the measured tensile strength [FEL 04].

It is revealed that a relatively large number of tests could be carried out under some of the thermal modes (1-7 above). However, to date most tests such as microscopy thin section analysis have usually been performed "residual" (i.e. after cooling). The same applies to most permeability tests that have been performed after cooling. Advances in research indicate that these restrictions would be overcome by novel instrumentation and test techniques.

Historically, the mechanical properties have been presented for concrete heated at constant temperature - even without loading during heating. This can be misleading because the influence of load during heating is very significant and large strain properties such as LITS develop which dwarf the strains measured at constant temperature.

The load regime could also vary such that heating could start under constant load - or the load applied at constant temperature – usually after the material has "stabilised". Varying load regime is also sometimes employed to simulate certain dynamic conditions. The time to relative stability depends upon the level of constant temperature, which in turn depends upon the transformation rate of the physico-chemical processes.

The moisture boundary condition could include:

- totally sealed specimens,
- semi-sealed specimens,
- unsealed specimens.

While it appears that, for fire applications, the unsealed condition is the only relevant mode, this is not fully correct. Although moisture escapes from the surface, it is also driven down the temperature gradient into the inner zone of the concrete. Depending upon the size of the member and the heating rate, moisture could accumulate to above the initial unheated level and significant pore pressures develop that could contribute to the spalling of concrete. The inner zone could, therefore, resemble sealed or semi-sealed moisture conditions, and the relevant thermo-hydro-mechanical properties would be different from those of unsealed concrete.

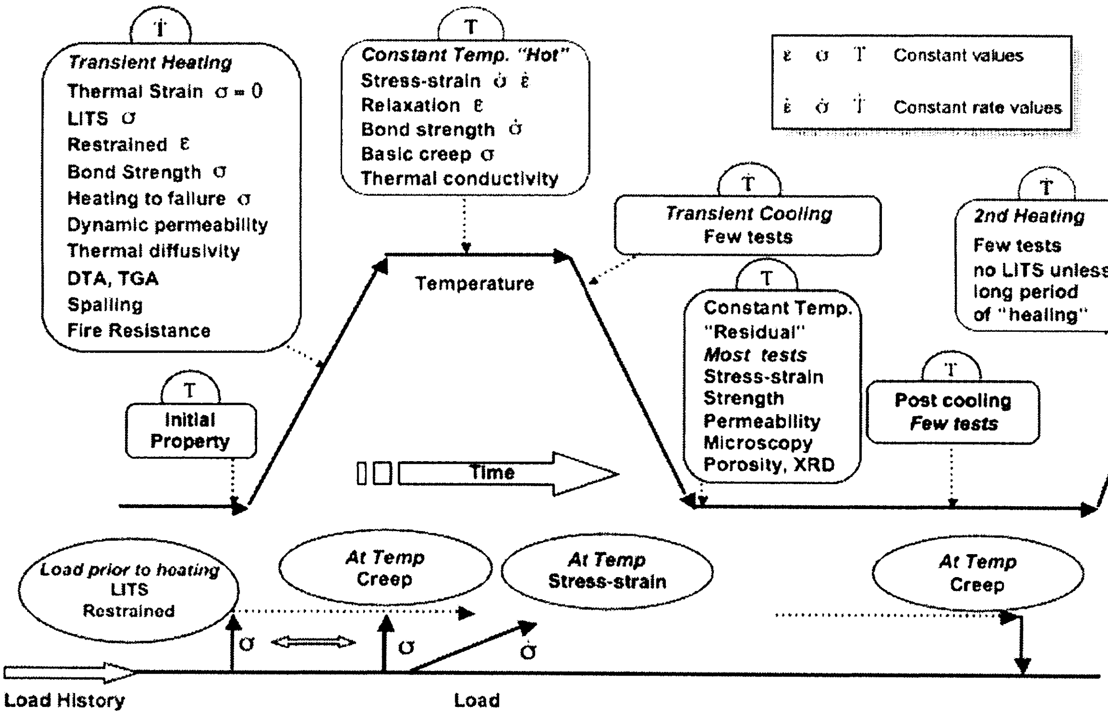


Figure 3.1. Examples of the thermo-hydro-mechanical properties of concrete dependant upon the load-heat-time sequence of the test regime [KHO-05]. Note: moisture boundary conditions are not shown .

Therefore, in summary, the time-loading-heating sequence and moisture boundary conditions are critical to the information produced by research and hence the information used by engineers and code committees. Often the wrong data is used because of lack of awareness of issues such as those raised in Figure 3.1.

3.4 Microstructure

In fire applications, the testing and practical use of thermal and mechanical properties alone may be insufficient. Although thermo-mechanical modelling has been used successfully in modelling the deformations and limit states of concrete structural elements, when it comes to concrete mix design against fire, it is essential to understand the microstructual behaviour of the material during fire – including the hydral condition. This means not only the cement paste but also crucially the aggregates – as well as the composite.

3.4.1 Importance of microstructure

Changes to the microstructure of concrete during heating result in corresponding changes in its properties. A direct clear link between the physico-chemical changes at the microstructural level of heated concrete and the thermo-hydro-mechanical behaviour of the material at the larger phenomenological level has not been easy to establish, partly because of the complexity of the science involved, the diversity of the types of concrete available and the largely dispersed and isolated nature of the tests that have been conducted. Understanding the microstructural behaviour of concrete during heating helps explain this linkage, provides a valuable design tool in respect of fire, and answers a number of interesting questions such as:

- Why does concrete normally "lose strength" for temperatures up to 80°C but "increase in strength" at higher temperatures up to about 200-300°C?
- What causes spalling, and how do PP fibres reduce the probability of spalling?
- What is "LITS" and why is it unique to concrete?
- Why has the aggregate such a considerable influence upon the thermal strain of concrete but hardly any influence upon LITS?
- Why the very large difference in the thermal strain (i.e. shrinking cement paste and expanding aggregate above 100°C) does not result in the total break-up of concrete?
- Why does the type of concrete influence its strength so much under hydrothermal conditions?

These intriguing phenomena, and others, are answered briefly in this and subsequent sections.

The starting point of understanding the cement paste is its constituents (Figure 3.2) and the physico-chemical process that takes place on hydration. One of the main products that influence the high temperature behaviour is the calcium silicate hydrate (C-S-H or Tobermorite) which is a colloidal structure having a very large surface area. Amongst the other products of hydration is the crystalline calcium hydroxide ($Ca(OH)_2$).

Heating concrete induces physico-chemical changes throughout the temperature range from ambient up to melting (Figure 3.22) thus influencing its thermal, hydral and mechanical properties. These changes occur primarily in the hardened cement paste, but to some extent they may also occur in the aggregate depending upon the type of aggregate used. Changes that take place in concrete at the "lower" temperature ranges (e.g. 20-300°C) reflect mainly changes in the cement paste since most commonly used aggregates are relatively stable up to about 300°C. Added to these are the physical and chemical interactions between the aggregate and cement paste. The cooling process also results in physico-chemical changes in concrete (e.g. crack development, moisture absorption, re-hydration of CaO). Therefore, the behaviour of the composite material (i.e. concrete) during both heating and cooling depends upon the:

- type of cement paste;
- type of aggregate;
- bond region
- interaction between them.

3.4.2 "Concrete type" and not "concrete"

For this reason, it is totally inappropriate to refer to "concrete" but rather to "**concrete type**" because in fire different concrete types (i.e. constituents, mix proportions, preparation etc.) can produce radically different behaviours. This is a commonly missed key and important point. **Concrete as a generic material does not exist when it comes to fire.**

3.4.3 Cement paste

The ingredients of hydrated cement paste are the chemicals shown in Figure 3.2.The cement paste itself will be influenced the by the mix proportions of these ingredients. So here for a starter, the cement paste can be open to design depending on the components and proportions therefore used in the mix.

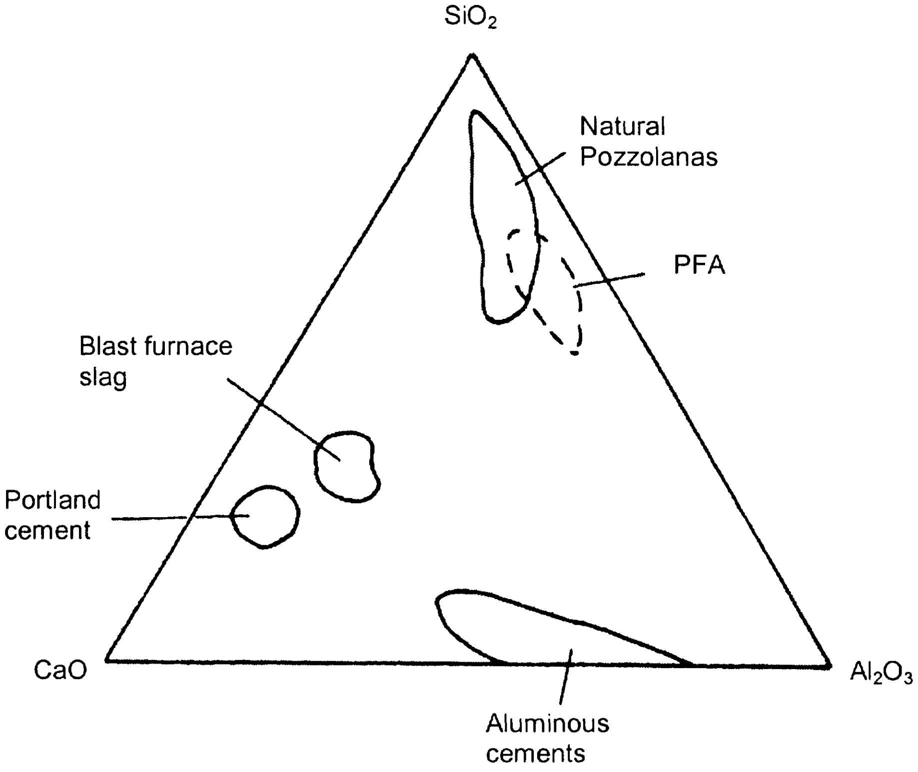


Figure 3.2: Cement paste is formed from different proportions of these constituents. For example a high content of slag or pfa would decrease the C/S ratio

With the addition of water the cement paste develops in stages which are also dependant upon not only the constituents involved but also upon the method and temperature curing. Ultimately, the high temperature behaviour would depend upon parameters such as the water/cement ratio, the C/S ratio, the amount of $Ca(OH)_2$ formed, and the degree of hydration amongst other factors. The purpose is to show from the outset that different cement pastes could be formed even before the aggregate is added and that different cement pastes can perform differently in fire.

Starting with these end products it would be interesting to follow the changes in physics and chemistry as the mature hardened cement paste is heated for the first time.

Any discussion on the microstructural behaviour of heated cement paste should start by distinguishing between the very different behaviours of "unsealed" cement paste (i.e. allowed to dry during heating) and "sealed" cement paste (i.e. retains moisture under hydrothermal conditions, particularly above 100°C). In fire, the heated surface region of concrete loses its moisture by evaporation from the surface and by moisture migration into the inner concrete mass driven by the temperature gradient (Figure 3.4). The surface region may, therefore, resemble "unsealed" concrete while at some distance from the surface the concrete may experience hydrothermal conditions depending upon a number of factors such as the heating rate and the permeability of the concrete. It is important to note that in concrete applications (e.g. buildings) with a relatively low initial humidity of say 50% prior to fire, the thermo-hydro issues are not as important as it would be in, for example, tunnels with higher initial humidities of say 75%. Therefore caution should be applied when considering the practical effect of moisture and pore pressures in relatively dry concrete. In both cases, explosive spalling could occur but it is more likely in humid tunnels.

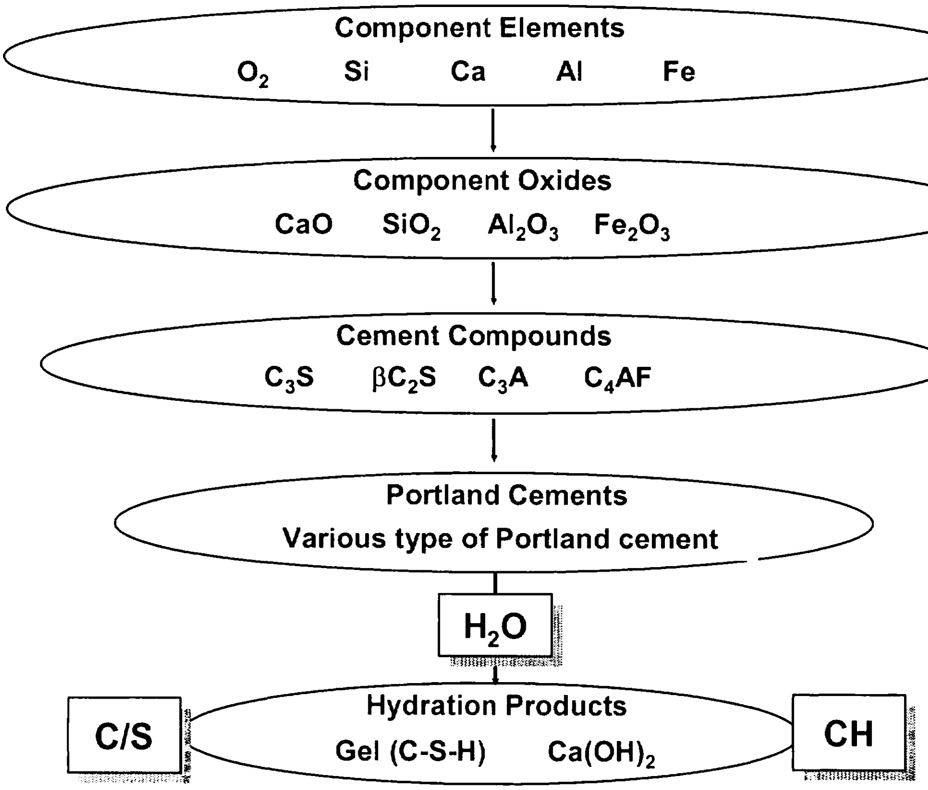
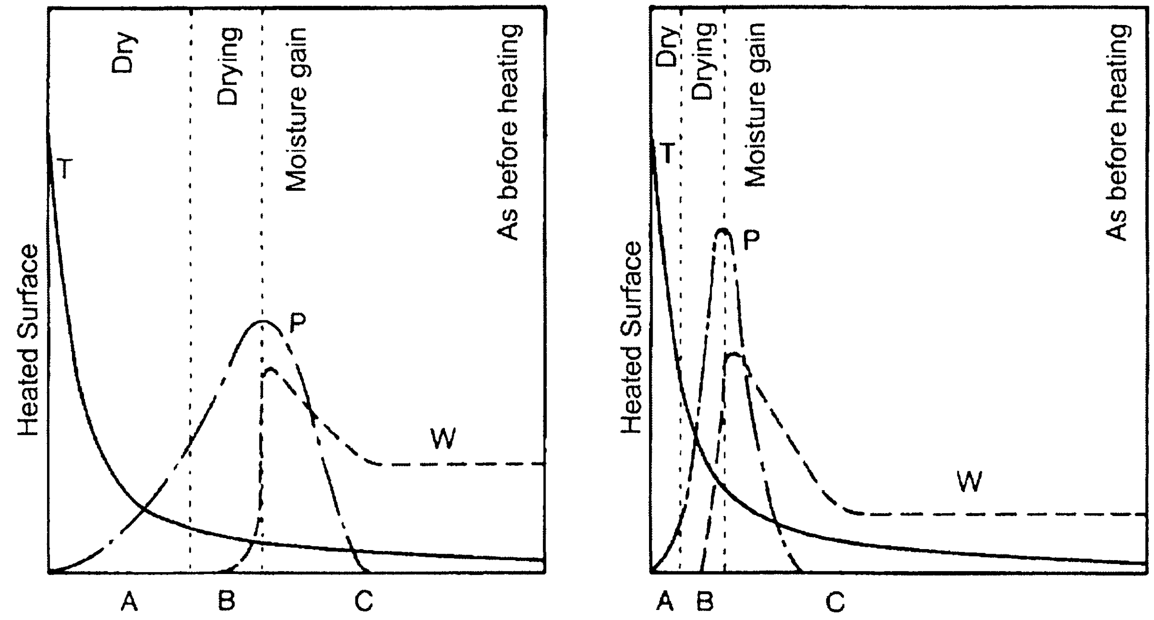


Figure 3.3: Hydration products formed after mixing with water produce Gel and $Ca(OH)_2$ amongst other products. Along with the C/S ratio, they influence the behaviour of cement and concrete during heating [KHO 05].

The cement paste is a very unstable constituent of concrete during first heating as it undergoes major physical/chemical transformations in both the sealed and unsealed conditions. It is generally accepted that within the range 4-80°C the hydration products of ordinary Portland cement (OPC) essentially remain chemically unaltered. Changes in properties within this temperature range can, therefore, be attributed to physical effects (e.g. changes in van der Waals cohesive forces, porosity, surface energy, crack pattern) or to accelerated hydration. Above about 80°C the microstructural behaviour of unsealed cement paste starts to diverge significantly from that of sealed cement paste. All the phase changes in unsealed cement paste (Table 3.1) are strongly influenced by temperature, and are strictly thermodynamically not fully reversible, although reversibility is often assumed in mathematical modelling [KHO 02a]. The changes in sealed cement paste depends very much on the C/S ratio (Figure 3.4). A non-exhaustive schematic diagram is given in Figure 3.5 showing the dissociation of components of cement paste and their re-association upon cooling, whereas Figures 1, 2 & 4 intend to illustrate in a simple format the chemistry involved in hydration, dissociation upon heating and re-hydration upon cooling relevant to fire applications.



a) Normal concrete

b) High performance concrete

Figure 3.4: Schematic diagram showing temperature (T), moisture (W) and pore pressure(P) states during heating in "normal" and "high strength" concrete.

3.4.3.1 Unsealed cement paste

Heating unsealed increases the mobility of water and moisture diffusivity of unsealed cement paste which accelerates drying and the loss of, first, free (capillary) water and then physically bound (adsorbed and interlayer) gel water. The chemical transformations occurring in unsealed cement paste are essentially those of **hydration** below 100°C and **dehydration** (e.g. of CSH and $Ca(OH)_2$) above 100°C, except when hydrothermal conditions occur temporarily in the inner regions of large members when chemical transformations could take place (Figure 3.4). The most significant microstructural changes that take place during heating of unsealed cement paste (e.g. primary chemical bonds, secondary cohesive [van der Waal] forces, porosity, pore size distribution, microcrackimg) are associated with the loss of physically and chemically bound water. This is evident in the region up to about 800°C within which the cement paste is held together by hydraulic bonds. It terms of design and analysis note should be given to the physical dilation of heated water, physical loss of water from large capillary pores accompanied by a negligible effect on porosity. Added to these physical effects is the chemical process of accelerated hydration. Other components of cement paste experience transformations at given temperature ranges measured by DTA (Differential Thermal Analysis) which measures indo- and exo-thermic reactions, and TGA (Thermo-gravimetric Analysis) which measures processes that result in weight loss. Added to these are dilatation tests, pore-size distribution tests, thin section analysis tests, SEM tests, XRD test and a whole plethora of microstructural tests that allow a much fuller understanding of the behaviour of heated cement paste.

Decomposition of the cement gel and loss of chemically bound water begins at about 80°C and chemical conversion of the gel continues non-linearly until it is 70% and 100% converted at about 500°C and 850°C respectively (Figure 3.6). The *rate* of this conversion reaches a peak at about 150°C and declines steadily till about 650°C when it increases again to reach a second but lower peak at about 720°C. Relating to the first peak at 150°C, it is not certain if the loss of chemical water actually contributes to strength loss of the material. Considering the chemical bonds prior to dehydration to be in the form of hydroxyl silanol groups (Si-OH HO-Si), it is not inconceivable that the loss of at least part of the bound water may lead to the formation of shorter, and stronger, siloxane bonds (Si-O-Si) with possibly higher surface energy, thus contributing to shrinkage and increase in strength [KHO 92]. This is in addition to the "gain" in strength resulting from the loss of gel water. However, in the second peak at about 700°C , CSH tobermorite gel decomposes into β-dicalcium silicate (β-C_2S), β-wollastonite (βCS) and water, which is in keeping with test results which consistently show a marked reduction in the strength (hot and residual) of concrete above 600°C. β-C_2S can be recognized as one of the main original cementitious compounds in cement which can rehydrate on cooling if exposed to moisture, thus leading to some restoration of bonds. Loss of chemically bound water causes an increase in porosity and coarsening of the pore structure due to the creation of additional pore space [ROS 80, PIA 84, KHO 92] thus contributing to decline in strength.

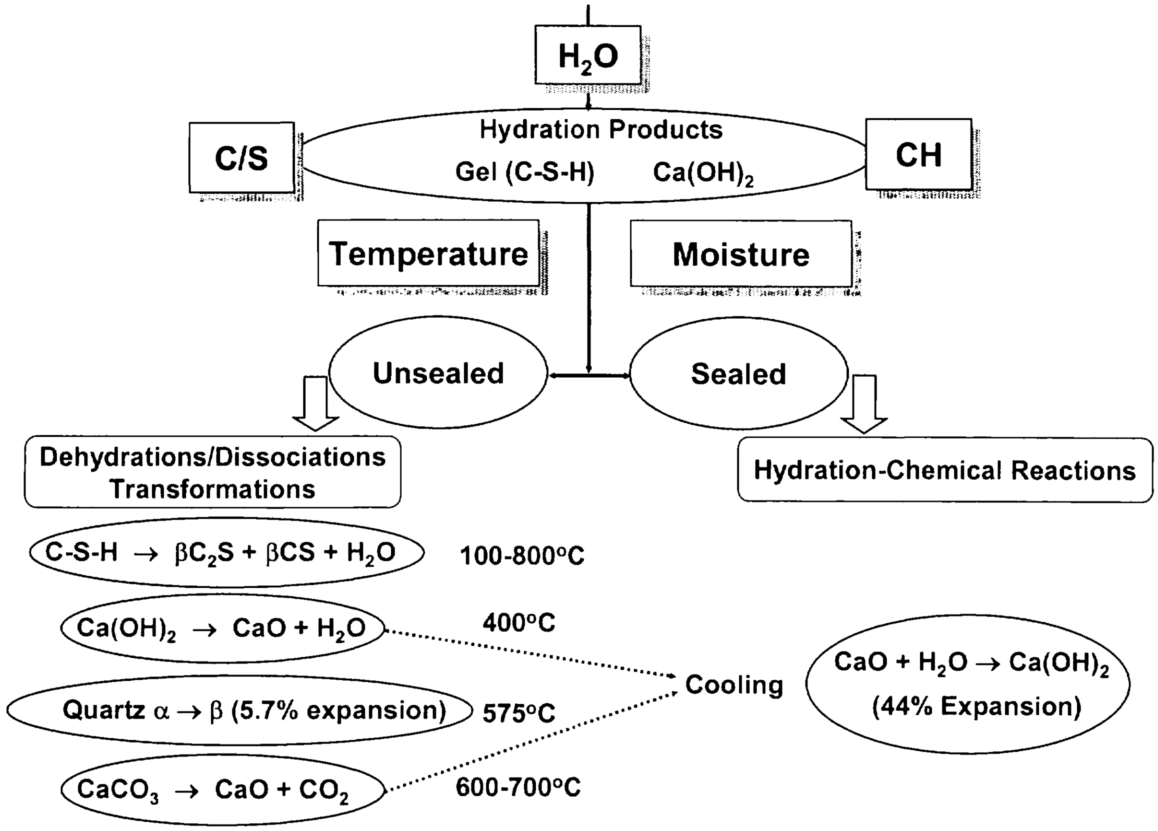


Figure 3.5: The influence of temperature and moisture (sealed and unsealed) upon the chemical transformation of hydration products [KHO-05].

Superimposed upon these transformations is the other major dehydration process, the dissociation of the $Ca(OH)_2$ into lime (CaO) and water (H_2O). The rate of decomposition is zero at about 400°C, reaches a peak at about 500°C and declines to zero rate at 600°C. The free lime that is formed reacts expansively with moisture on cooling thus causing a volume increase in excess of 40% with subsequent disruption of the bonds in cement paste. From this point of view $Ca(OH)_2$ is not a desirable component of cement paste. DTA of hardened Portland cement paste indicates that the transformations in unsealed cement paste are largely endothermic (Figure 3.7). A broad endotherm commences at a temperature less than 100°C reaching a maximum at slightly above 100°C due to the dehydration of various phases in hydrated cement as well as the continuing loss of evaporable water. Tobermorite gel and hydrated calcium sulphoaluminate are the first solid phases affected. The prominent endotherm between 500 and 600°C is due to the decomposition of $Ca(OH)_2$. At about 700-

800°C an endotherm is sometimes observed due to the decomposition of calcium carbonate ($CaCO_3$); this may be present as a result of carbonation by atmospheric carbon dioxide of the $Ca(OH)_2$ formed during hydration of Portland cement.

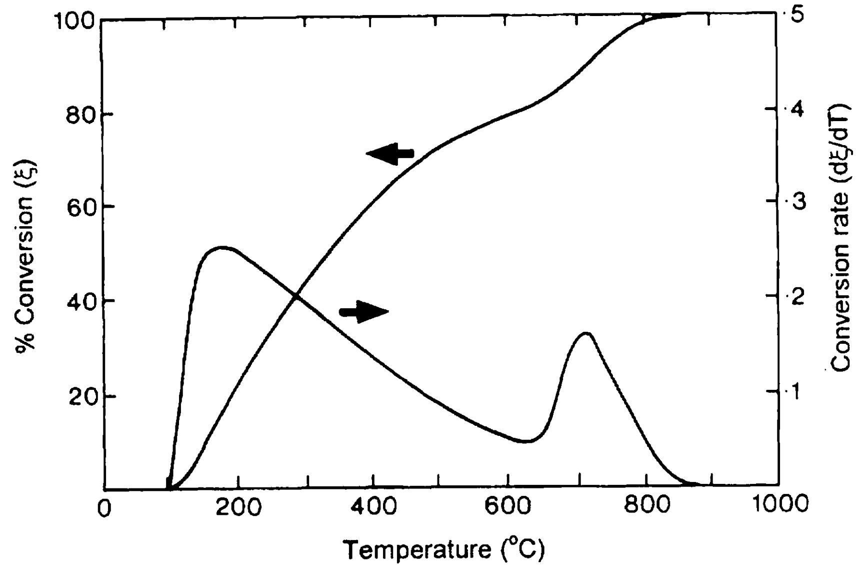


Figure 3.6: Degree are rate of conversion of Tobermorite gel (C-S-H) in an idealised heated cement paste – excluding loss of free water [HAR 70].

The total **porosity** and especially the pore structure distribution are important features that influence a key parameter – permeability. The total porosity of cement paste comprises the full spectrum of pores spanning from the relatively large capillary pores (>2000Å) down to the fine gel pores (20Å). The overall porosity of cement paste increases non-linearly with temperature due to the progressive breakdown of the gel structure of CSH as dehydration proceeds. However, pores of various sizes do not necessarily increase in the same proportion and some can decrease with increase in temperature. Generally, the increase in total pore volume after heating from 20°C to 300°C is smaller than expected from weight loss measurements [ROS 80]. This effect is more pronounced in OPC than blast furnace slag paste indicating that changes in pore structure at high temperature depend on the cement blend used. Although the increase of the total pore volume from 20°C to 300°C remains rather small, a significant change is visible in the differential distribution (Figure 3.8). The maximum at 20°C, located at a pore radius of about 550Å (450Å for blast furnace slag cement) is flattened by the influence of temperature and shifts towards larger pore sizes. Heating from 300°C to 600°C leads to a significant increase of the total pore volume which is greater than one would expect from the additional loss of weight beyond 300°C. Scanning electron microscopy indicates that microcracking increases significantly beyond 300°C, first around $Ca(OH)_2$ crystals and then around unhydrated cement grains [PIA 84], and particularly around larger grains. In comparison with the cumulative pore size distribution at 300°C there is an almost parallel shift upwards at 600°C, thus signalling the coarsening of the pore structure in the range greater than 5,000Å and a new maximum appears in the coarse pore range of about 15,000Å /452/. At 850°C porosity of the paste exceeds the value at 105°C by about 40%. High temperature treatment up to 900°C results in a further increase of pore volume. The differential distribution shows two maxima, one at about 700Å, the other at about 20,000Å while the portion of small pores below 200Å has decreased. This may possibly be caused by the onset of the sintering process. It should be noted, however, that these results were determined by means of mercury intrusion porosimetry (MIP) measuring pore sizes down to 40 Å. The marked reduction in the strength of OPC paste observed above 300°C (Figure 3.9) would, therefore, at least in part be explained by the significant increase above this temperature in both average pore size and microcracking.

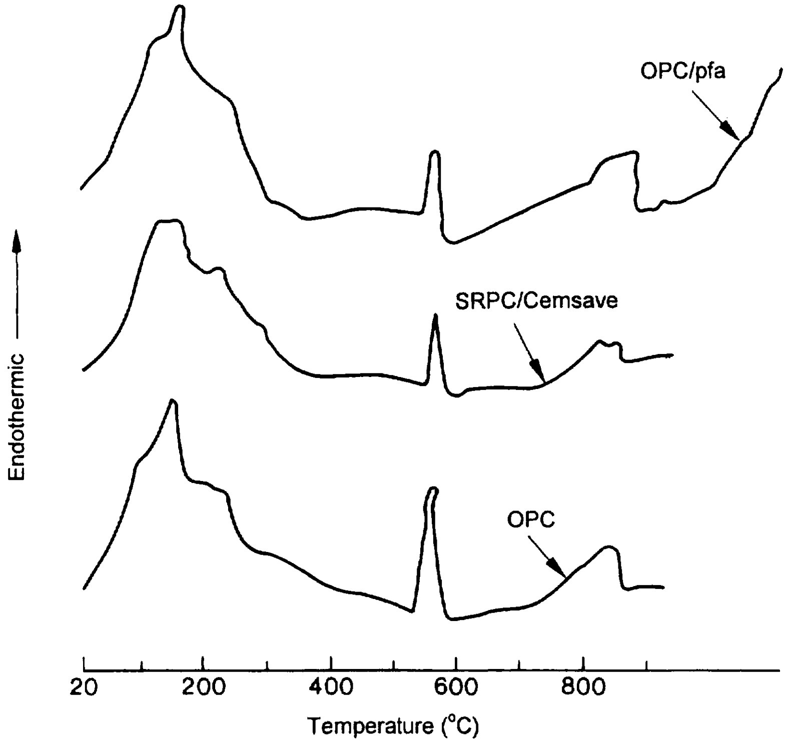


Figure 3.7: Differential thermal analysis (DTA) of three cement blends heated at 10°C/minute in nitrogen [KHO 85b]. OPC = ordinary Portland cement. Cemsave is granulated blast furnace slag, SRPC is sulphate resisting Portland cement, pfa = pulverised fuel ash.

Heating materials increases molecular thermal agitation and reduces surface energy resulting in strength reduction and leading ultimately to melting. Although this effect is negligible in cement paste at the lower temperature ranges it becomes significant at temperatures above about 550-600°C. Such "**Thermal**" effects become significant above about 500-600°C causing a ten-fold surge in "basic creep" [KHO 86a] suggesting a viscous or molecular diffusion type phenomenon similar to that observed in other materials (e.g. glass, ceramics, and metals) above certain critical temperatures. This phenomenon is reversible in so far as the creep of concrete previously heated to 650°C but subsequently cooled to, and loaded at, 595°C produced creep comparable to a similar specimen heated only to 595°C [KHO 86a]. "*The important implication of this effect is that rheological criteria limit the structural, though not necessarily the insulating, usefulness of Portland cement based concretes in general to temperatures about 550-600°C*". However, in fire, the bulk of the concrete may not reach such temperatures although exposed surface regions may experience higher temperatures.

Ceramic binding takes place in cement paste at about 800°C at which temperature all chemically bound water would have been lost. Ceramic binding causes an increase in residual, though not hot, strength, and is employed in aluminous refractory concrete where it replaces hydraulic binding after first firing [ROB 78].

Ultimately cement paste starts to **melt** at temperatures in excess of 1,100°C depending upon its chemical composition [BAZ 96] . In general, ordinary Portland cement paste starts to melt at about 1350°C.

Loss of chemically combined CSH and $Ca(OH)_2$ water results in non-linear reduction in *weight* above 100°C. Only a marginal reduction in weight takes place above 850°C as dehydration at this temperature is complete. During first heating, cement paste experiences a slight expansion below 100°C as free water escapes, followed by significant *shrinkage* above this temperature (Figure 3.11) as physically and chemically bound water escape, and then reaching a plateau between 500°C and 800°C or expanding again depending upon the proportion of unhydrated cement grains present in the paste. This is another example of the clear linkage between the material behaviour (e.g. shrinkage) and the underlying operative mechanisms.

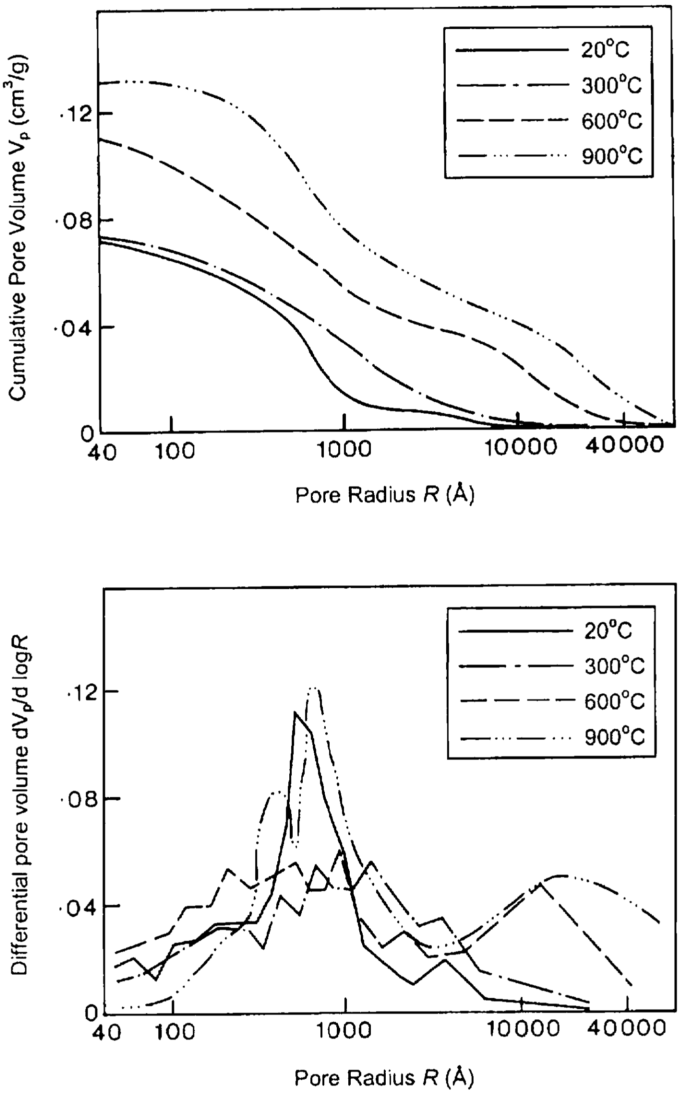


Figure 3.8: Effect of heating on cumulative pore volume and pore size distribution of unsealed Portland cement paste [PIA 84].

The behaviour of unsealed cement paste during, and after, cooling is dominated by the absorption of moisture from the surrounding medium. Changes that take place during heating are to a certain degree irreversible, although rewetting of the cement paste after cooling can lead to some restoration of bonds partly through rehydration. However, the CaO produced during previous heating from the dehydration of $Ca(OH)_2$ (above 400°C) and decarbonation of the $CaCO_3$ (above 700°C) absorbs water on cooling and rehydrates into $Ca(OH)_2$ with a corresponding 44% increase in volume. This expansion causes the development of cracks and weakening of the paste. Although this is an undesirable effect, after a fire any concrete previously subjected to temperatures in excess of about 300°C is normally removed and replaced.

Another finding given in Figure 3.10 indicated "high strength concrete" (HSC) is micro-structurally just an extension of "normal" strength concrete, whereas "ultra high strength concrete" (UHSC) behaves micro-structurally very different as evidenced by its much slower weigh loss during first drying at 105°C (Figure 3.10). UHSC was investigated under the HITECO multi-national EU project. The slow weight loss owes to the nature of the materials preparation and its compactness thus reducing premeability significantly even in relation to HSC. Although the terms of high performance concrete (HPC) and ultra high performance concrete (UHPC) may be appropriately used for normal service conditions against chemical hazards, when it comes to fire the reverse is true. HPC and UHPC concretes become poor performers in fire (unless maybe when they include PP fibres). For this reason, in this report, the terms high strength concrete (HSC) and ultra high strength concretes (UHSC) are used as the preferred terms.

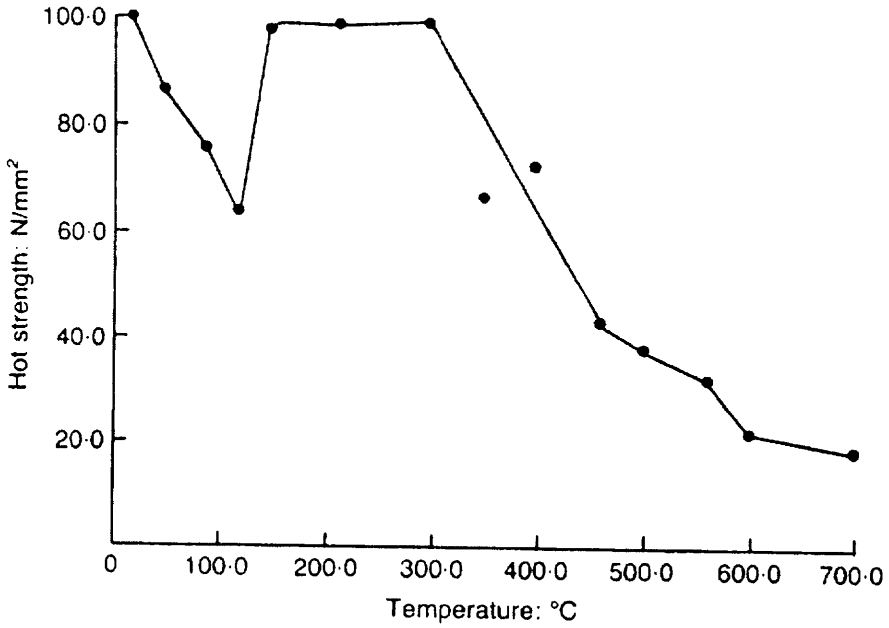


Figure 3.9: Compressive strength of pure unsealed cement paste specimen against temperature when tested "hot". The "apparent" loss in strength at about 100°C is due to the dilation of heated moisture and reduction in the van der Wall's physical bonds [DIA 90]

3.4.3.2 Sealed cement paste

Heating relatively immature sealed cement paste tends to accelerate hydration and improve strength, but at temperatures above 80-100°C any hydration of cement paste could be beneficial or detrimental to strength depending on the C/S ratio of the gel. It is generally accepted that within the range 4-80°C the hydration products of OPC remain essentially chemically unaltered. Changes in properties within this temperature range can, therefore, be attributed to physical effects (e.g. changes in porosity, surface energy, crack pattern) or to accelerated hydration.

When cement paste is heated above 80-100°C in the moist sealed condition, hydrothermal reactions take place which can result in significant changes in its chemical/physical microstructure. The nature of the phase changes that take place depend on the C/S ratio, temperature and pressure levels (Figure 3.12). The C/S ratio of mature Portland cement paste may vary as widely as 1-3 but the mean is close to 1.5 [YOU 82]. Heating cement paste with a C/S of about 1.5 to temperatures in excess of about 110°C produces a weak C_2S-hydrate (also termed $C_2SH(A)$). The structure is highly porous and crystalline, of low specific surface area, with correspondingly reduced cohesive (van der Waal) forces. However when a cement paste with a C/S ratio of about 1.0 is heated to 150°C under sealed conditions, a strong tobermorite gel is formed. Clearly, it is important to ensure that cement paste exposed to hydrothermal conditions contains a low C/S ratio. This is achieved by adding silica in a reactive form in appropriate quantities which also increases the amount of CSH produced. At the optimum silica content of 30%, the 'autoclaved' cement paste is significantly stronger than in the normally cured paste [MEN 34]. In practice, silica can be exist in concrete in the form of slag, pfa, silica fume or siliceous fine aggregate. The addition of 65% slag causes a 30% increase in strength of mature cement paste when heated in hot water at 140°C for four weeks, while the strength of ordinary Portland cement paste reduces by about 5-25% [MAR 78]. Improvements in strength of sealed dolomitic concrete at temperatures 100-150°C can be obtained with 25% replacement of cement by pfa. However, at temperatures above 180-200°C, other different products are formed such as Xonolite and Hillebrandite which also depend on the C/S ratio [NAS 79].

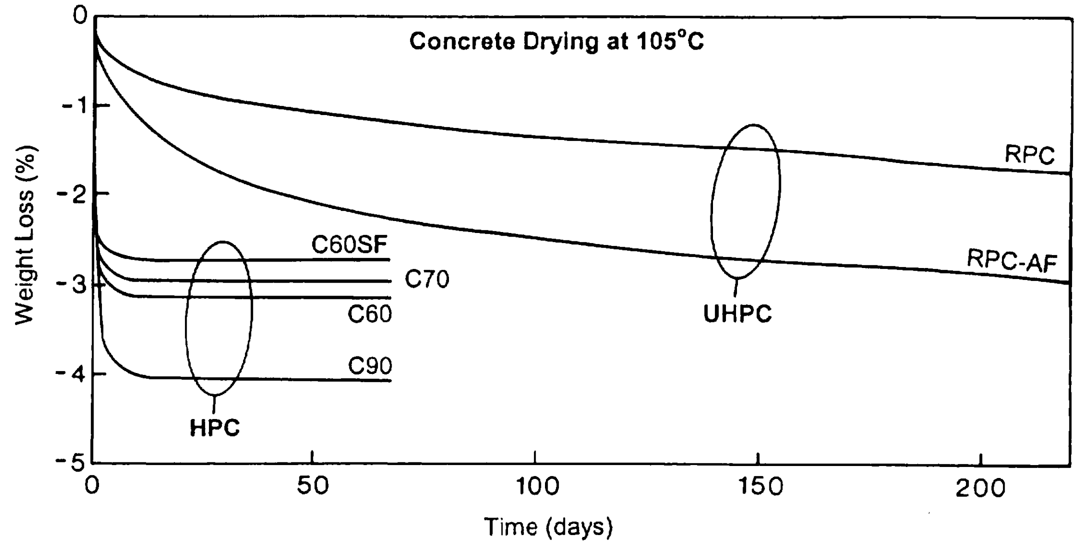


Figure 3.10: Slow weight loss of unsealed UHSC concrete when compared with to HSC concrete during drying at constant 105°C, indicating very different micro-structures [ALO 99]

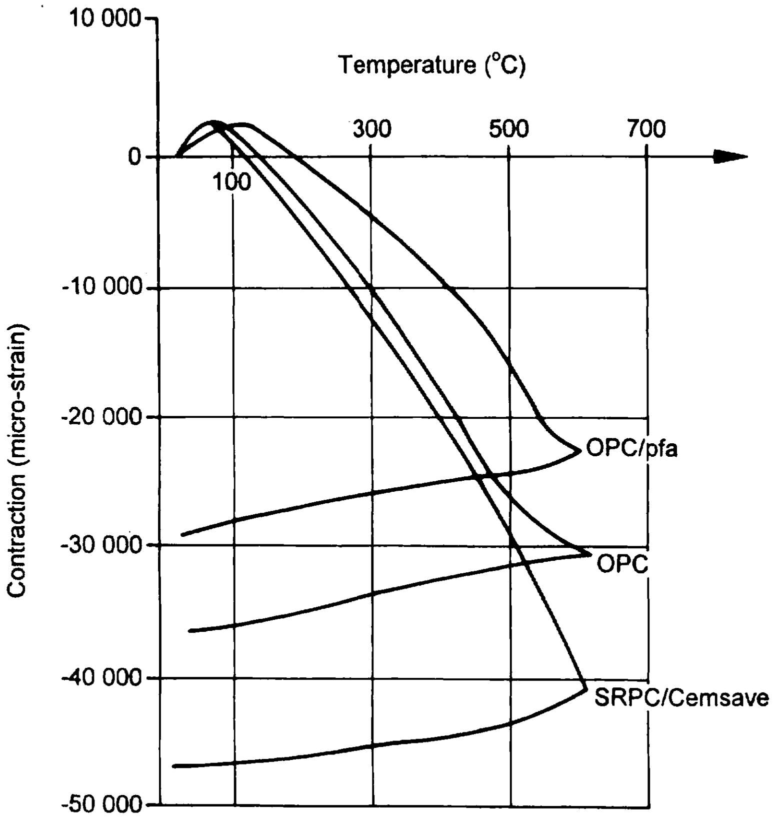


Figure 3.11: *Thermal strain of three cement pastes during first heating at 2°/min to 600°C and during subsequent cooling [KHO 85b].*

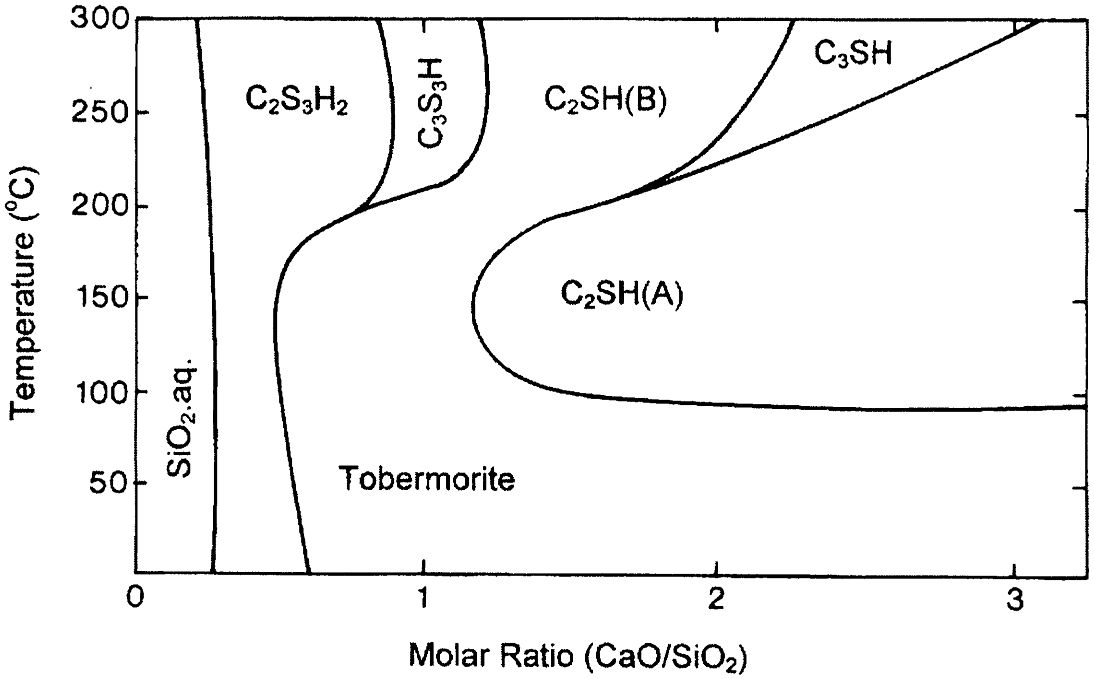


Figure 3.12: Influence of the C/S ratio and temperature level upon the products on the development of calcium silicate hydrate C-S-H (including tobermorite) hydration products [KOTT 79].

3.4.4 Aggregate

Given the large emphasis placed in research upon the cement paste behaviour, the poor relation has always been the aggregate in terms of mix design. This guideline emphasises the key role that the aggregate plays and that it really should be considered from the outset in the mix design. A few reasons of why the aggregate is so important follows:

- It occupies some 60-80% by volume of concrete
- Variations in aggregate properties on heating can have a significant effect on the performance of concrete at elevated temperatures. The thermal strain and the thermal conductivity of concrete, for example, are to a large degree governed by those of the constituent aggregate.
- Physico-chemical changes also occur in the aggregate depending upon the type of aggregate used, but aggregates differ greatly in their response to heat. A key factor in the behaviour of heated concrete is the chemical and physical stability of the aggregate. The choice of aggregate is, therefore, an important factor in determining the thermal properties, and thermal stability, of the concrete in fire.
- Restrains creep and shrinkage of the cement paste.

It is not appreciated that aggregates differ markedly in their thermal stability. Examples are given in Figures 3.13-3.16 of thermal stabilities of a range of aggregates.

Thermal stability of the aggregate is measured by thermogravimetric (weight loss) tests, differential thermal analysis (revealing thermic reactions – Figure 3.14), and dilatometry (Figure 3.15), including residual strain tests (Figures 3.16). Aggregates that are thermally stable up to a given temperature show no weight loss, no thermic reactions and negligible residual strain.

Lightweight aggregates, particularly manufactured ones, such as expanded slags and cinders, exhibit high chemical and physical stability at temperatures even exceeding 800°C since they are normally manufactured at around 1,300°C. These coupled with a low coefficient of thermal expansion and thermal conductivity should rank lightweight aggregates among the best in terms of temperature resistance. However, it should be noted that the water present in the relatively porous lightweight aggregates (e.g. 50-60% porosity) in high moisture environments (e.g. some tunnels) would increase the risk of explosive spalling. The opposite is true in dry environments where lightweight aggregate concrete would have a lower risk of spalling than normal weight concretes owing to the low thermal expansion of the aggregate. Lightweight aggregates are sometimes recommended for use in concrete against spalling but the moisture condition must be taken into account.

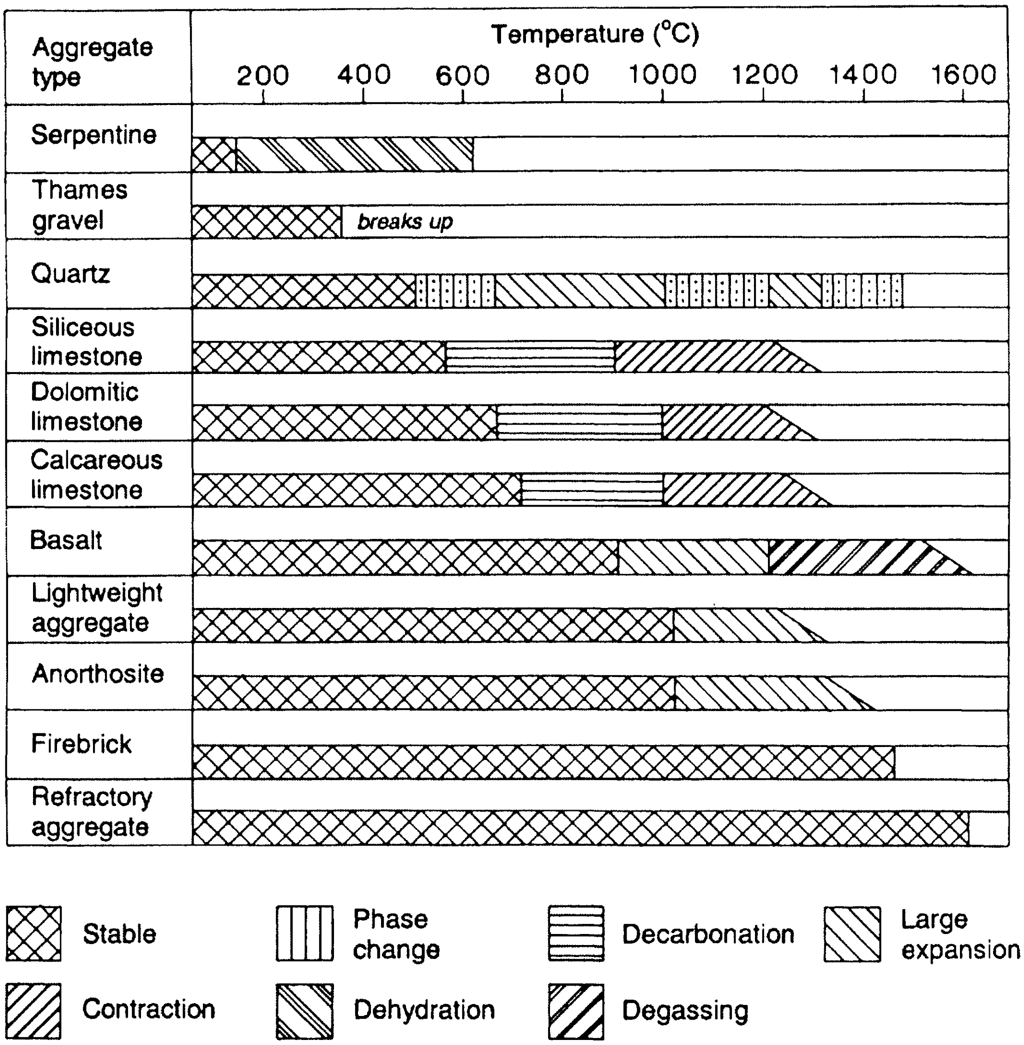


Figure 3.13: Non-exhaustive examples of the large range of thermal stabilities, and processes, that take place in aggregates during heating [KHO 05].

Quartz (SiO_2) present in siliceous aggregates and sands is subject to a number of physical changes at elevated temperatures (Figure 3.14). The best known is the reversible endothermic (5.9kJ kg^{-1}) crystalline α-β transformation of quartz which peaks at 575°C accompanied by a sudden volumetric expansion of about 5.7%. These volumetric changes could have damaging effect on concretes made with such materials when exposed to high temperatures although some experts have questioned whether in fact these transformations lead to severe damage to the concrete or produce spalling. Such aggregates include river gravel, sandstones and quartzite rocks.

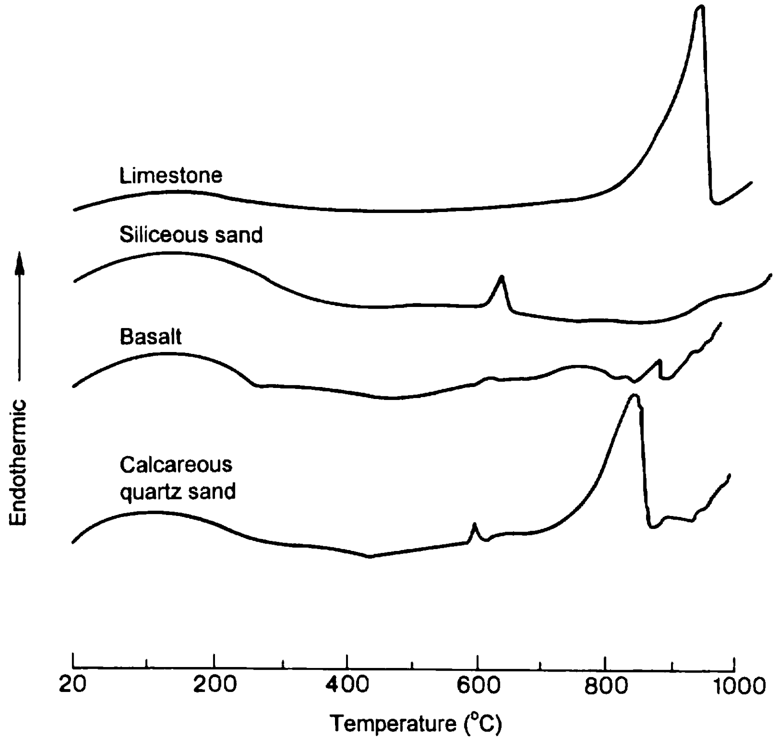


Figure 3.14: Physico-chemical transformations in four different aggregates as measured by DTA during heating at 10°C/min in nitrogen [KHO 85b]

Note: α–β transformation of quartz shown in the peak at 575°C, de-carbonation is manifest in the larger peaks of the limestone and Calcareous aggregates.

Carbonate aggregates such as limestones and dolomites are stable up to about 700°C when the calcium carbonate ($CaCO_3$) starts to decompose into lime (CaO) and carbon dioxide (CO_2). This process expels a considerable amount of CO_2 and reaches a peak at about 800°C and is completed at a pressure of one atmosphere at 898°C. The rate at which the reaction takes place depends not only on the temperature and pressure but also on the amount of silica [BAZ 96]. The dissociation of calcium carbonate at high temperatures is endothermic and tends to retard the temperature rise in concretes made with such materials when exposed to fire. This extends the period of fire resistance but deterioration is likely to occur on cooling when the free lime combines with atmospheric moisture to form $Ca(OH)_2$. Magnesium carbonate aggregates similarly decompose between 740°C and 840°C releasing carbon dioxide and absorbing heat in the process. In both cases the material becomes less dense and hence a better insulator. Furthermore, the carbon dioxide, escaping from the surface of the concrete in considerable volume, forms an inert insulating layer thus further retarding temperature rise of concrete. Such aggregates are also sometimes called calcareous aggregates.

Aggregates produced from crystalline rocks may be liable to progressive disintegration at high temperatures because of thermal incompatibility within the aggregate itself and also possibly due to the dehydration of any chemically combined moisture present. The parent material may be composed of a variety of closely packed minerals whose differences in thermal expansion can cause inter-crystalline stresses and failure. Thames river aggregate, for example, expands and considerably breaks up at about 350°C because of this effect (Figure 3.15) and concrete made with it also disintegrates.

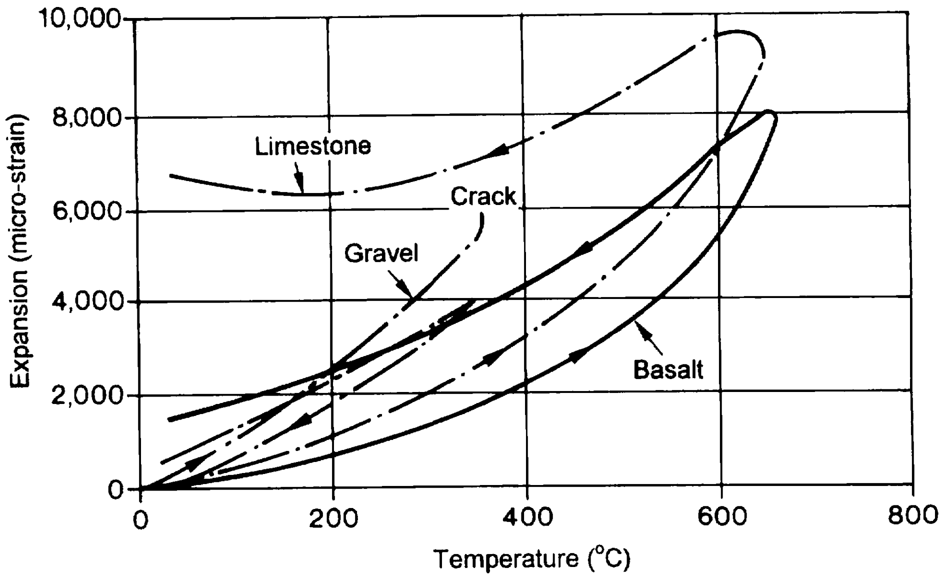


Figure 3.15: *Thermal strain during the first heat cycle (2°C/min) of three rectangular aggregates (20-30mm) showing the break-up of Thames river gravel at 350°C, and the lower thermal stability of the limestone than the basalt [KHO 85b]*

Aggregate porosity varies from nearly 0% to 50% or more for some lightweight aggregates. Water present in aggregate pores is evaporable and can be expelled upon heating thus contributing to the build up of pore pressures in concrete. Hydrous aggregates (e.g. serpentine, limonite, goethite) which contain chemically bound water of crystallization (e.g. 4-12% by weight), sometimes used in concrete for radiation shielding, have dehydration temperatures ranging from 175°C to 500°C depending upon the aggregate type [KAP 89]. Moderate dehydration also occurs in some siliceous or calcareous aggregates holding some water of constitution which is accompanied by shrinkage[WAU 73] (about 13% in opal at 373°C).

The melting temperatures of aggregates reported by different authors vary greatly (e.g. basalt 1060°C, dolerite 1070°C, granite 1210-1260°C, quartzite 1700°C, pure CaO 2570°C). During melting some aggregates (for example, igneous rocks such as basalt) expand and release gases that were trapped within the rock at the time of its formation [BAZ 96].

In sealed concrete, hydrothermal reactions between the $Ca(OH)_2$ in the cement paste and magnesium carbonate aggregates tend to weaken the concrete. Reaction of any reactive silica present in the siliceous aggregate and sand with the $Ca(OH)_2$ can lead to stronger bonds forming in the concrete. Therefore, the roles of carbonate and siliceous aggregates can to a certain extent be reversed in the case of sealed concrete heated to temperatures up to about 200°C.

In terms of fire resistance [SMI 78], crushed firebrick and fused aluminium oxide (corundum) and other special aggregates used in refractory concretes are at the top of the list. Then in descending order of fire endurance come expanded slags, shales, slates, and clays; air-cooled slag; basic, finely grained igneous rocks such as granite and basalt; calcareous, siliceous aggregates and lastly flint. Recent tests at temperatures up to 600°C show Gabbro from Finland to be a thermally resistant aggregate (Figure 3.15).

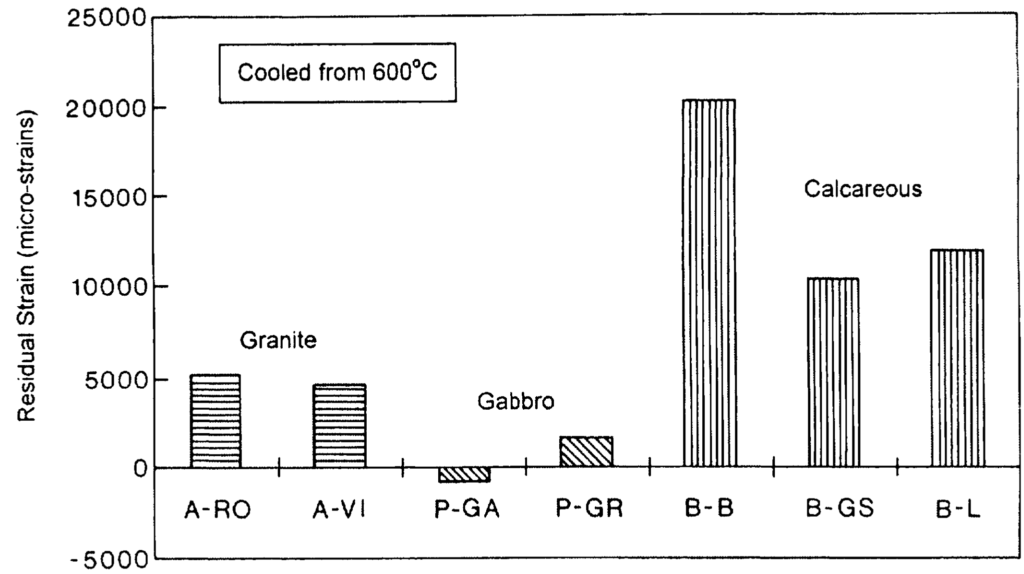


Figure 3.16: Residual strains of different aggregates indicating increasing thermal stability of gabbro, granite, and calcareou aggregates [KHO 97].

3.4.5 Bond region

The aggregate-cement paste bond region has been shown to be the weakest link in the concrete chain since it is normally weaker than the cement paste, which in turn is normally weaker than the aggregate [ALE 68]. A "bond shell" forms around the aggregate with properties different from the bulk of the cement paste. The reactivity of the aggregate can be beneficial to bond strength or harmful to bond depending upon the nature of the chemical reaction taking place. Autoclaved concretes employing quartz aggregate were found to attain compressive strengths in excess of 150 N/mm^2 partially attributed to quartz-cement paste reaction producing a bond stronger than the cohesion of the paste itself [MAS 80]. This is an important result in that it indicates that the bond "shell" is not destined always to be the weakest link in the chain. It also demonstrates that heating can in certain cases produce an improvement in the bond strength. It can also be attributed to the expansion of the aggregate relative to the cement paste adding a compressive stress on the bond interface.

A faint whitish rim was found around the aggregates of a limestone concrete, giving a dense strong concrete at ambient temperatures. However, after heating to 300°C, the chemical changes in the minor constituents and particularly the iron oxides leads to almost complete destruction of this bond [CAM 66]. Some deterioration in bond occurred on the smooth surfaces of expanded shale aggregate, but with fire brick clay negligible change was found [CAM 66].

Thermal stability of the aggregate alone is, therefore, not sufficient to produce concrete which maintains its original strength at high temperatures. The aggregate should also produce a strong temperature "resistant" bond with the cement paste.

Studies on bond strength at ambient temperatures have shown that, everything else being equal, variations in bond strength can result in concrete strength variations of 300% or more [NEP 69, LEZ 67]. For example, lubricating the aggregate can cause a reduction in concrete strength by as much as 2.5 times [NEP 69], while coating the aggregate with polymer has increased concrete strength by a factor of 1.5-2.0 [LEZ 67]. These results suggest that if the aggregate-cement paste bond fails on heating, chemically or as a result of thermal incompatibility between the aggregate and cement paste, the concrete will exhibit a significant reduction in strength, even if both the aggregate and the surrounding mortar matrix remain intact.

3.4.6 Interaction between the aggregate and cement paste

Interaction between the aggregate and cement paste can be physical (e.g. thermal incompatibility) or chemical.

3.4.6.1 Physical interaction

Physical interaction takes the form of differential thermal expansion between the aggregate and cement (Figure 3.17) resulting in the build up of "parasitic" stresses that can lead to the weakening and disruption of the concrete at elevated temperatures. During first heating, cement paste experiences a slight expansion as it is heated up to 80-100°C while free water escapes. This is followed at higher temperatures by significant shrinkage as physically and chemically bound water escape. By contrast, most commonly used aggregates continue to expand above 100°C while the paste shrinks. A large differential thermal strain develops that cannot be accommodated by elastic and basic creep strains alone. On this basis concrete should indeed begin to break down at about 100°C as have been predicted earlier [LEA 20, LEA 22]. Yet the threat does not seem to materialise even in some concrete samples heated to 600°C, largely because of the considerable load induced thermal strain (LITS) component which appears during first time heating of cement paste under compressive load [KHO 06a] and which to a large extent accommodates the differential thermal strain between the expanding aggregate and shrinking cement paste. This would explain why some researchers [LAN 71] found no difference in the crack pattern of the cement matrix in gravel concrete heated to 260°C compared with as cured concrete. Crack patterns observed in concrete after cooling differ depending on the mix constituents, loading condition, and the rate of cooling, and it is possible to observe increased micro-cracking in the cement paste after cooling from a temperature as low as 40°C, but the damage would have been much greater in concrete heated to temperatures above 100°C had LITS not existed.

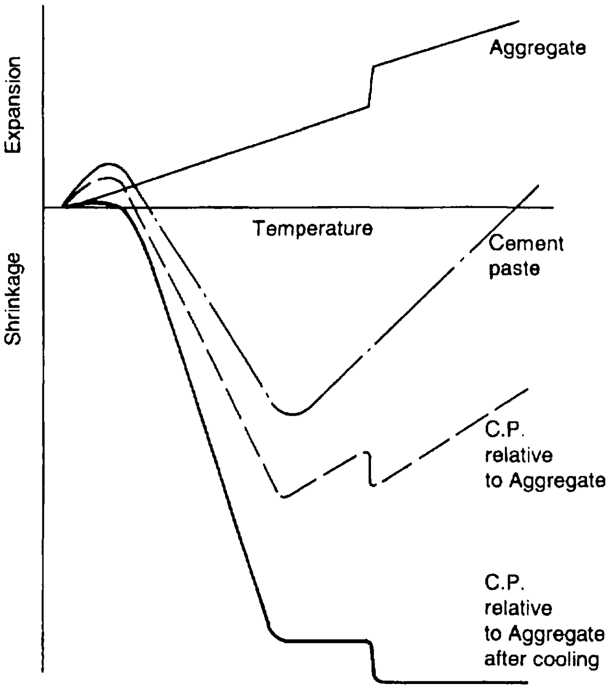


Figure 3.17: Schematic diagram showing thermal incompatibity between the aggregate and the cement paste (c.p.) during first heating.

Note 1: the cement paste could go into a plateau, or expand again at higher temperatures depending upon the size and amount of un-hydrated cement grains.

Note 2: An elastic analysis [LEA 22] suggests that concrete should break up at 100°C on account of this thermal incompatibility - but it does not owing to the influence of LITS in compression [KHO 06a]. However, cement paste does not experience LITS in tension [HAG 05].

Supposing concrete is heated uniformly so that the traditional thermal stresses are zero, the interparticle stresses would then be some function of the difference in free thermal movements of the aggregate and cement paste. These "parasitic" thermal stresses could be tensile in the cement paste if it shrinks with respect to the aggregate. Their magnitude vary with the type of aggregate. On first heating the differential strains can initially induce a small compressive stress in the cement paste but with a further increase in temperature this is reduced by LITS in compression, although LITS does not appear in tension [HAG 05]

During cooling the thermal incompatibility would be accommodated by elastic and basic creep strains only since LITS does not take place during cooling. Consequently the problem of thermal incompatibility during cooling could be more serious than during first-time heating. Assuming the interparticle "parasitic" compressive stresses generated during first-time heating are largely relaxed by LITS, they could develop during cooling depending on the relative contraction of the aggregate and mortar and upon the mechanical properties of the two constituents as well as upon the rate of cooling. The potential problem posed by thermal incompatibility during first heating is, therefore, different in character from that encountered during cooling and subsequent re-heating.

3.4.6.2 Chemical interaction

An example of chemical interaction, influenced by temperature, is the reaction that takes place between the $Ca(OH)_2$ crystals released by hydration of ordinary Portland cement and the magnesium carbonate component of some limestone aggregates;

$$MgCO_3 + Ca(OH)_2 \rightarrow Mg(OH)_2 + CaCO_3 \qquad \text{eq 3.1}$$

This reaction is expansive and causes the weakening and disruption of the concrete. It could, however, be prevented by the addition of pozzolanic material, such as pfa to the concrete mix [RAA 74]. Preferential reaction of $Ca(OH)_2$ with silica in the ash leaves the aggregate unscathed and an additional amount of calcium silicate, the binding constituent of the cement, is formed.

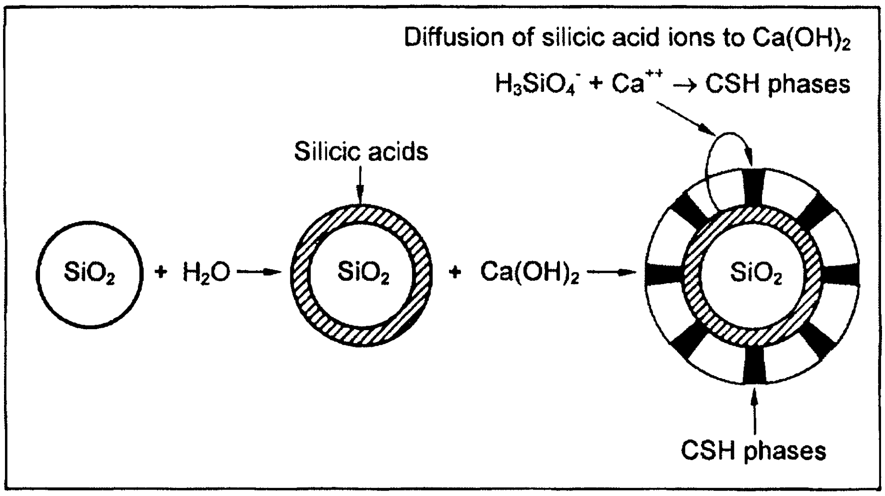


Figure 3.18. Schematic presentation of the hydrothermal reaction between reactive silica (SiO_2) and $Ca(OH)_2$ [KOT 79]

3.4.6.3 Heating concrete can have an enhancing and not just degrading function

There has been a proliferating tradition in publications to assume that all the dehydration processes and transformation processes above about 80°C essentially cause degradation. This may not be entirely true as described above. This thinking has led the proliferation of "damage theories" related to heated concrete. The foregoing discussion shows that "thermally enhancing functions" also exist at certain temperature ranges and under certain load-moisture-heat conditions. For example, such knowledge has lead to steam curing and autoclaving as methods to stabilise the precast concrete and reduce subsequent shrinkage and creep (Figure 3.19).

Thermally stable aggregates are used to attain greater stability of the concrete. Polypropylene fibres are used in the concrete mix to reduce pore pressures during heating thus reducing the risk of spalling. Essentially these, and other measures, are taken to improve the behaviour of concrete in fire by addressing one of more weakness.

Take the marked apparent reduction in "strength" of unsealed cement paste at 80-100°C (Figure 3.9). This is essentially a reversible physical phenomenon. The reduction in strength results from the dilation of the adsorbed water and consequent weakening of the cohesive forces and reduction in surface tension. While the increase in strength is experienced above this temperature (in both unsealed concrete and much more in cement paste) results from the reversal of these phenomena. Scientist have been intrigued by this behaviour.

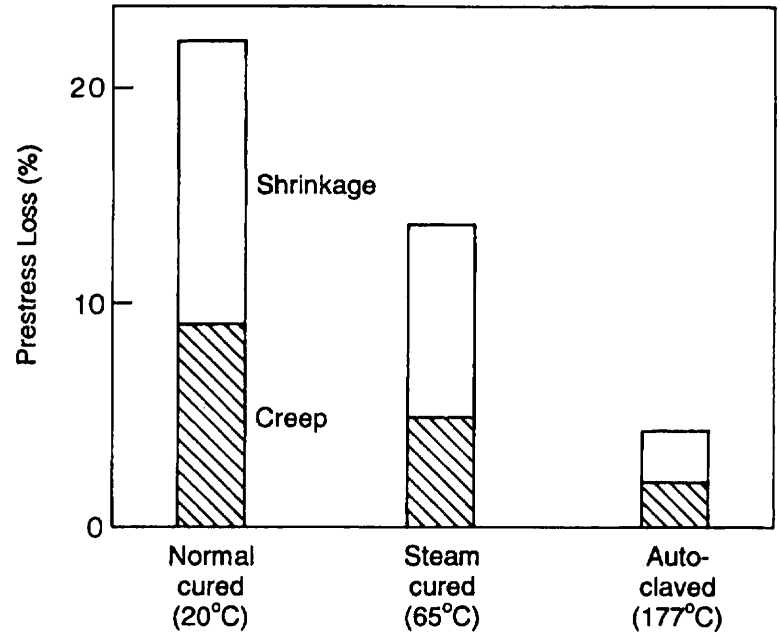


Figure 3.19: Schematic presentation of the hydrothermal reaction between reactive silica (SiO2) and (Ca(OH)₂) [KOT 79]

Measures that reduce the size of unhydrated cement grains, and the choice of cement blend which reduces the $Ca(OH)_2$ content (e.g. by employing pfa or slag as part replacement of OPC) could, therefore, lead to improved performance above 300°C (Figure 3.20). Both the dehydration of the CSH and $Ca(OH)_2$ produce a reduction in the solid skeleton mass of the cement paste. However, the strength of concrete is more related to the calcium silicate hydrate (CSH) than the $Ca(OH)_2$.

Even a fair understanding of the microstructual changes in concrete experienced during fire can assist in solving problems that these changes might bring about, thereby allowing solutions to be explored, and thus demonstrating that not all processes encountered upon heating cause degradation. Some are "enhancing".

3.4.6.4 Post cooling thermal history assessment

A very old and well established technique has been to examine the colour changes that take place in heated concrete as shown in Figure 3.20. This applies to siliceous concretes. In

Ulster University, a thermoluminescence technique has been developed to ascertain the previous history of the sand in the concrete. These two examples are mentioned for interest only.

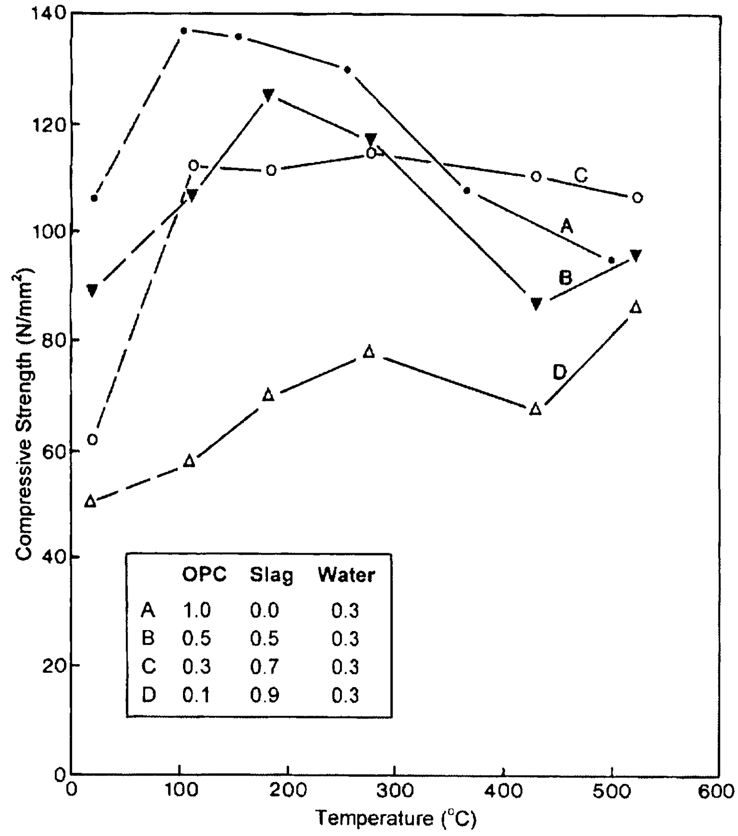


Figure 3.20: Improvement of the temperature function of strength of OPC paste with the addition of slag [KHO 92]

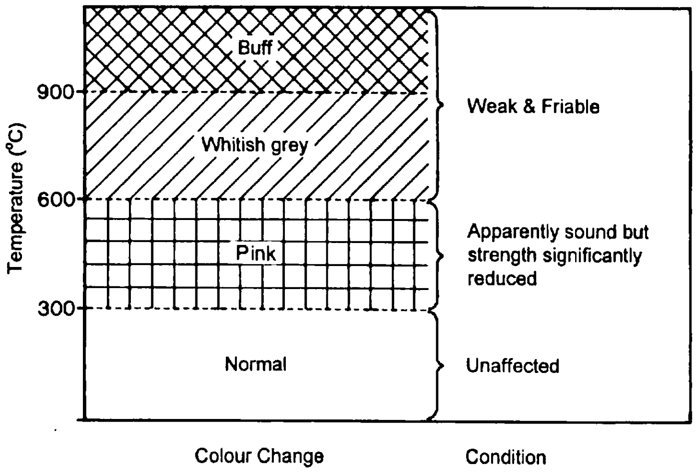


Figure 3.21: Colour changes in heated siliceous concrete.

Having covered briefly the behaviour at the microstructural level of the various constituents of concrete including the physico-chemical interaction, it would be of interest to present a simplified graphical example of processes taking place in unsealed concrete in terms of elevated temperature range to melting in the form of the "thermometer" analogy presented in Figure 3.22 with more detail given in Tables 3.1 and 3.2.

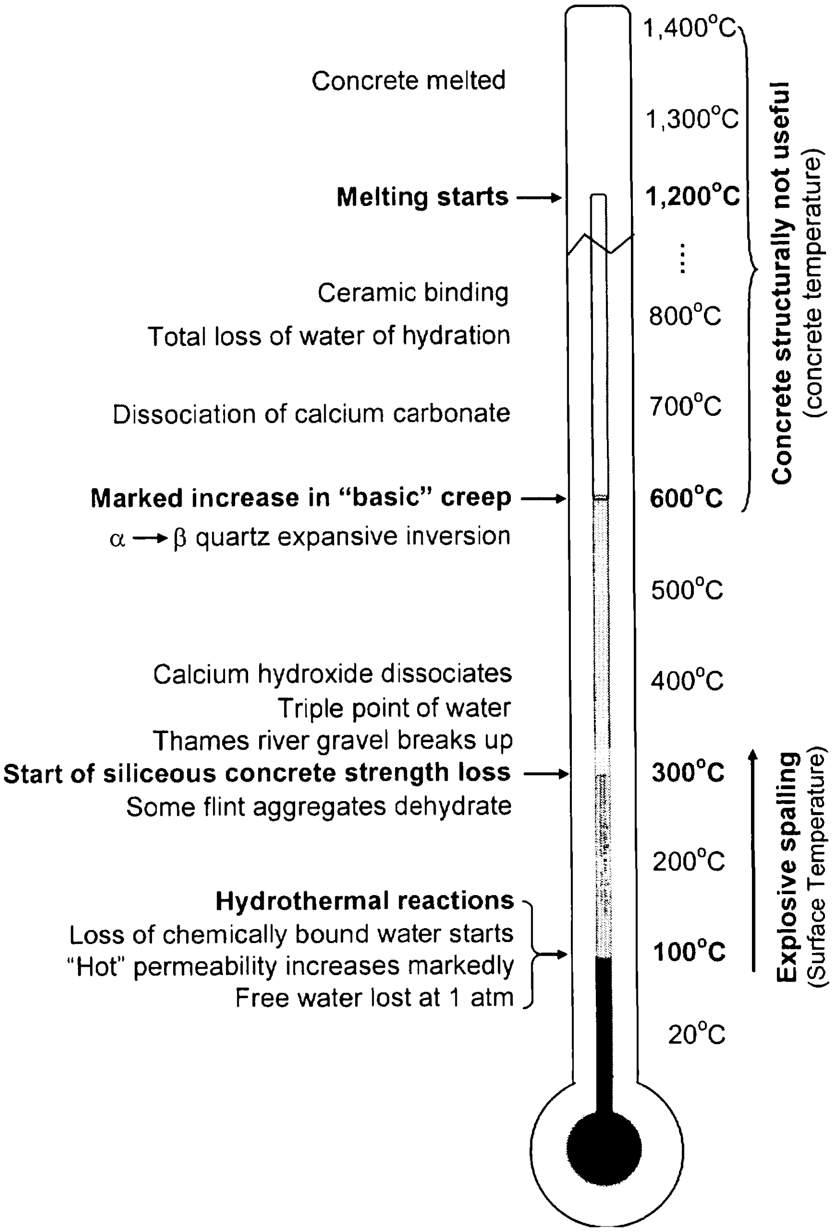


Figure 3.22. Simplified global presentation of physico-chemical processes in Portland cement concrete during heating in a "thermometer" analogy – for guidance only [KHO 00]

Notes: Temperatures given above are for the concrete material and not the fire, except for explosive spalling where the temperature range is that of the concrete surface and depends upon the heating rate and type of concrete.

Unsealed cement paste (i.e. allowed to dry) behaves very differently from moist sealed cement paste. Above 100°C, the former is dominated by dehydration processes, while the latter is dominated by hydrothermal chemical transformations and reactions.

Phases	Process	Energy	Phase Change Process
Dehydration Hydration	Chemical	loss gain	solid matrix (CSH or Ca(OH)2)⇒water-energy+matter (C or CyS) water+matter (C or CxS)⇒solid matrix (CSH or Ca(OH)2) +energy
Evaporation Condensation	Physical	loss gain	capillary water ⇒ water vapour - energy water vapour ⇒ capillary water + energy
Desorption Sorption	Physical	loss gain	physically adsorbed water ⇒ water vapour - energy water vapour ⇒ physically adsorbed water + energy
Decarbonation$ Carbonation	Chemical	loss gain	calcium carbonate ⇒ CO2 – energy CO2 ⇒ Calcium carbonate + energy
α–β inversion# β–α inversion	Physical	loss gain	Endothermic during heating up at 573°C with expansion Exothermic during heating down at 573°C with contraction

$ occurs in both cement paste and carbonate aggregates above 600°C.
# occurs in the quartz aggregates and sands

Table 3.1: Brief description of some phase changes in heated concrete [KHO 02a]

Temperature (°C)	Transformation
20-80	Increase in hydration, slow capillary water loss and reduction in cohesive forces as water expands
100	Marked (hundred-fold) increase in water permeability
80-200	Increase in the rate of loss of capillary water and then physically bound water
80-850	Loss of chemically bound water
150	Peak for the first stage of decomposition of calcium silicate hydrate (CSH)
300+	Marked increase in porosity and micro-cracking
350	Break-up of some river gravel aggregates
374	Critical point of water when no free water is possible
400-600	Dissociation of Ca(OH)2 into CaO and water
573	α–β transformation of quartz in aggregates and sands
550-600+	Marked increase in thermal effects
700+	Decarbonation of CaCO3 into CaO and CO2 (in paste and carbonate aggregates)
720	Second peak of CSH decomposition into β-C_2S and β--CS
800	Start of ceramic binding which replaces hydraulic binding
1,060	Start of melting of some constituents

Table 3.2: Brief description of transformations during heating of unsealed concrete [KHO 92].

4 Thermal properties

Fire assessment and design calculations must include thermal analysis, as a separate process, or as a process integrated with concomitant hydral and/or mechanical analysis. Thermal analysis is required for simplified or complex calculations. It is also required for analysis of separating or load-bearing function. A key to the success of thermal analysis is the appropriate choice, and use, of the thermal properties.

The first important step in the design process is the prediction of the temperature distribution of a fire-exposed structural member. The reliability of computed temperature profiles depends upon accurate and sufficient knowledge of the thermal properties of the building materials used. The thermal properties normally employed in computation are thermal conductivity (λ) and volumetric specific heat (ρc_p). The latter is simply the product of the density and constant pressure specific heat. It is these three properties that need to be determined experimentally for a given concrete or judicially selected from the literature/standards. They are required for the solution of the second order partial differential equation based on the principle of conservation of energy and given in its most general form by:

$$\rho c_p \, \partial T / \partial t = \text{div}\,(\lambda \text{ grad} T) + H \qquad \text{(eq 4.1)}$$

Where

λ = thermal conductivity (W/mK)

c_p = specific heat J/kgK

ρ = density (kg/m^3)

ρc_p = volumetric specific heat J/m^{3}K

H = rate of heat generated internally (e.g. heat of hydration, electrical source)

Without internal heat generation, equation 4.1 reduces to the Fourier head conduction equation.

In general it is a difficult if not an intractable task to solve partial differential equations analytically and in all but simple cases some numerical method is necessary for rendering a solution satisfying given boundary conditions.

4.1 Thermal diffusivity

The thermal diffusivity (D) gives a measure of the rate of heat flow under *transient* thermal conditions, and hence of the facility with which concrete can undergo temperature changes. It is described as follows:

$$D = \lambda / \rho c_p \quad (\text{m}^2/\text{s}) \qquad \text{(eq 4.2)}$$

Ironically, the thermal diffusivity of concrete is much easier to measure experimentally than its three components separately, but λ and ρc_p are required to be input separately into the calculations and this is where the problem lies. For example, the thermal conductivity at a given temperature is normally measured under steady-state temperature conditions, which is not representative of the unstable state of concrete under dynamic heating conditions. Similarly, time and temperature dependent endothermic and exothermic transformations (e.g. latent heat of evaporation, dissociation of calcium hydroxide) also dynamically influence the measurements of the specific heat at a given temperature. Given such difficulties, and the different equipment/procedures used in different laboratories around the world, wide variations of thermal properties can be produced for even apparently similar concretes, let alone for different types of concrete. Careful judgement therefore needs to be exercised in the choice and use of thermal properties directly from tests, or from the literature.

The thermal diffusivity of normal weight concrete falls markedly with increase in temperature as shown in (Figure 4.1) due to the general decrease in thermal conductivity and increase in the specific heat at elevated temperatures. Since the specific heat changes only modestly with temperature, except when latent heat is being absorbed, the temperature dependence of diffusivity shows a trend similar to that of thermal conductivity. The time dependent reactions that affect the specific heat of concrete on heating should reflect in corresponding transient variations in the thermal diffusivity or concrete. The fall in diffusivity with temperature is expected to be less steep for amorphous and lightweight aggregate concrete due to the slight increase in their thermal conductivity with temperature.

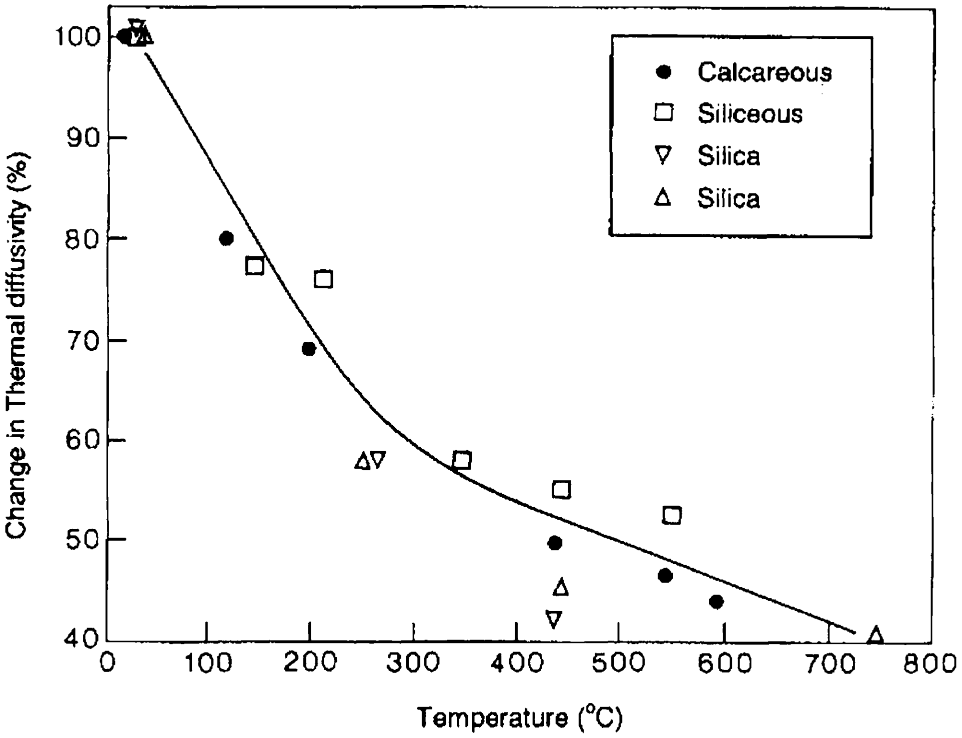


Figure 4.1: Effect of temperature on the thermal diffusivity of normal weight concrete excluding latent heat effects. Compiled from two sources [HAR 67], [HRD 72].

4.2 Thermal conductivity

Thermal conductivity was first defined by Fourier in 1822 as the ratio of the heat flux to temperature gradient. It is sometimes calculated indirectly from transient diffusivity measurements, particularly for moist concrete, since the latter is easier to measure. Direct conductivity measurements are complicated by the interaction between moisture and heat flow, and the results obtained depend on the method used [NEV 95]. It is not uncommon that conductivity measurements obtained from various labs on ‘identical’ specimens vary by a factor of two [COL 75]. One must, therefore, be careful before accepting data on the thermal conductivity of concrete. Direct thermal conductivity measurements are carried out at steady-state and not at transient temperatures. Physico-hydro-chemical transformations would be completed at the test temperature. Due to the uncertainty inherent in measuring thermal conductivity, numerical simulation of fire tests are carried out to compare measured temperatures over a cross-section with computed values based on measured thermal conductivity and by a fixed volumetric specific heat curve. If a discrepancy exists as normally it does, the values are modified by trial and error until a good agreement is attained.

Although the thermal conductivity of concrete depends on that of all its constituents it is largely determined by the type of aggregate used since aggregates normally constitute 60-80% by volume of concrete. The other important influence is the moisture content since the thermal conductivity of water, although low, is much higher than that of air (Table 4.1).

1 Material	2 Thermal Conductivity (W/m°C)
Aggregates	0.7-4.2
Saturated concretes	1.0-3.6
Saturated hardened cement paste	1.1-1.6
Water	0.515
Air	0.0034

Table 4.1: Thermal conductivities at ambient temperature [KHO 83]

The mix proportions and cement content therefore influence the thermal conductivity of concrete because of the disparity between the conductivity values of cement paste and aggregates (Table 4.1). A richer mix, therefore, in ordinary weight concrete has a lower thermal conductivity than a leaner one and the reverse is true for lightweight aggregate concrete. Increasing the water/cement ratio increases the concrete porosity and correspondingly reduces the thermal conductivity, especially when dry, since both water and air have lower conductivity values than cement paste. Although the thermal conductivity of concrete is little affected by temperature in the usual ambient range [NEV 95] it can significantly change at higher temperatures (Figure 4.2) owing to the physico-chemical structural changes that take place in concrete upon heating.

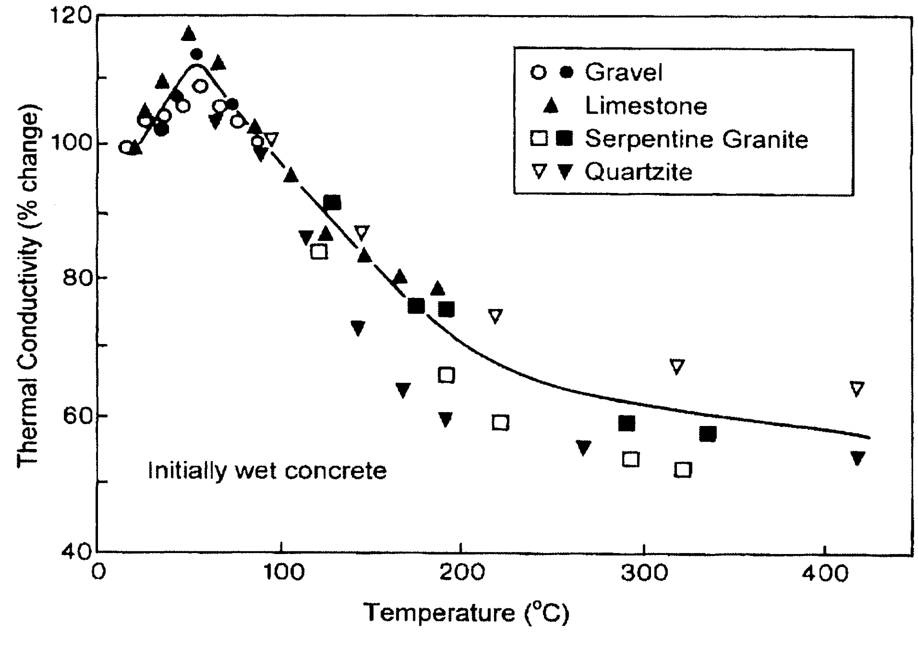


Figure 4.2: Effect of temperature on the thermal conductivity of initially saturated concrete [BLU 76].

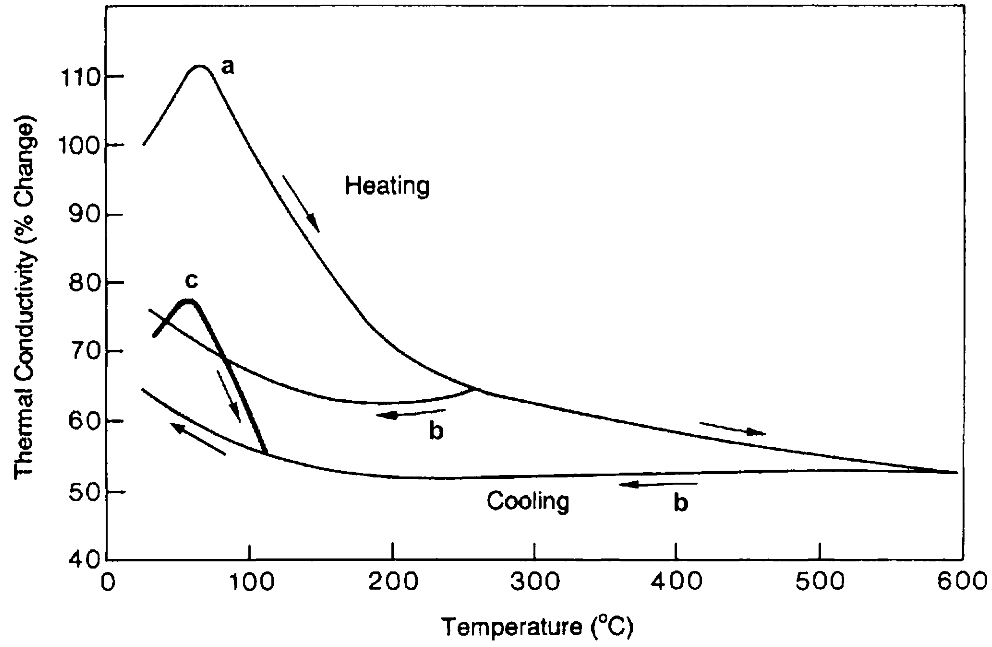


Figure 4.3: Effect of first heating (a) and cooling (b) and subsequent heating (c) on the thermal conductivity of normal weight concrete compiled from three sources [MAR 72], [CRI 72], [HRD 72]

The conductivity on cooling depends on the temperature level at the start of cooling. Figure 4.3 illustrates the general pattern of thermal conductivity variations on cooling. It can be seen that although the conductivity can increase by as much as 10-20% on cooling, most of the change is irreversible. After cooling to ambient, conductivity falls to a level still below the original unheated value. On re-heating, the conductivity measurements followed curve (a-b) in Figure 4.3 reaching a maximum at 50-60°C but at 105°C the curve returned back to the original cooling line. This indicates that the increase in thermal conductivity with temperature of rewetted cooled concrete is due to the presence of water and not to any significant restoration of conductivity bonds. First heating, therefore, leads to a change of the state resulting from physico-chemical transformations. Further heat treatments only bring about slight variations in relation to temperature as long as the initial maximum temperature is not exceeded.

4.3 Density

The changes in density of concrete are related to weight changes, thermal dilation and changes in porosity. Figure 4.4 for cement paste shows the variations of true density, bulk density and porosity. In the unsealed condition, these reflect the influences of the physico-chemical transformations that include water dilation up to about 80°C, loss of free and physically bound water at 100-200°C depending on the section size and heating rate, followed by the loss of chemically bound water for temperatures above 100°C, the dissociation of calcium hydroxide at about 400-500°C, and de-carbonation above 600°C. In concrete, the role of the aggregate is important in terms of thermal dilation and the dissociation of some aggregates such as carbonate aggregates that display a very significant reduction in density above 600°C and a marked increase in porosity. In basalt and quartzite aggregates exhibit a more gradual reduction in density reflecting that of the cement paste and dilation of the aggregate (Figure 4.5)

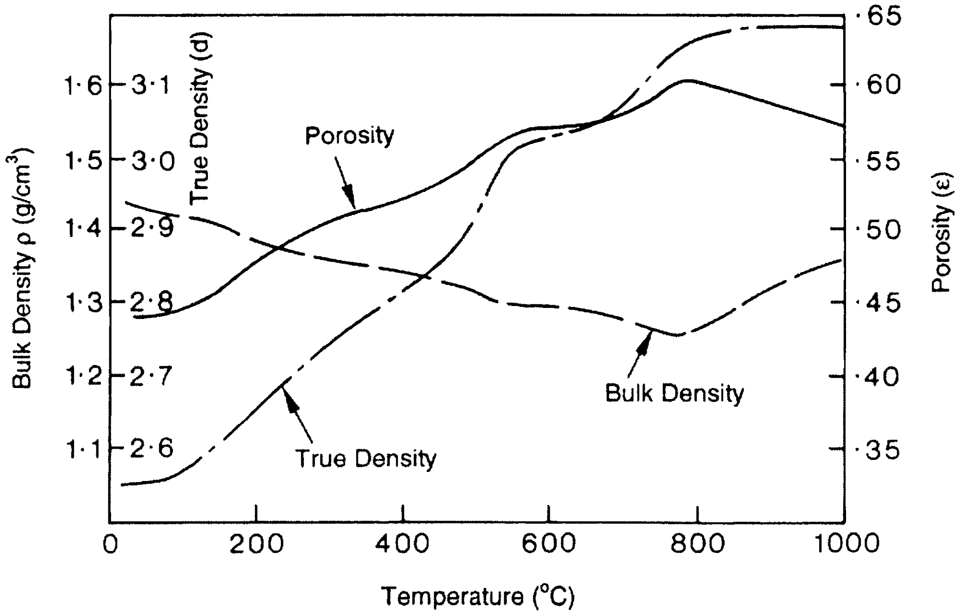


Figure 4.4: True density, bulk density and porosity of cement paste against temperature calculated by Harmathy [HAR 70].

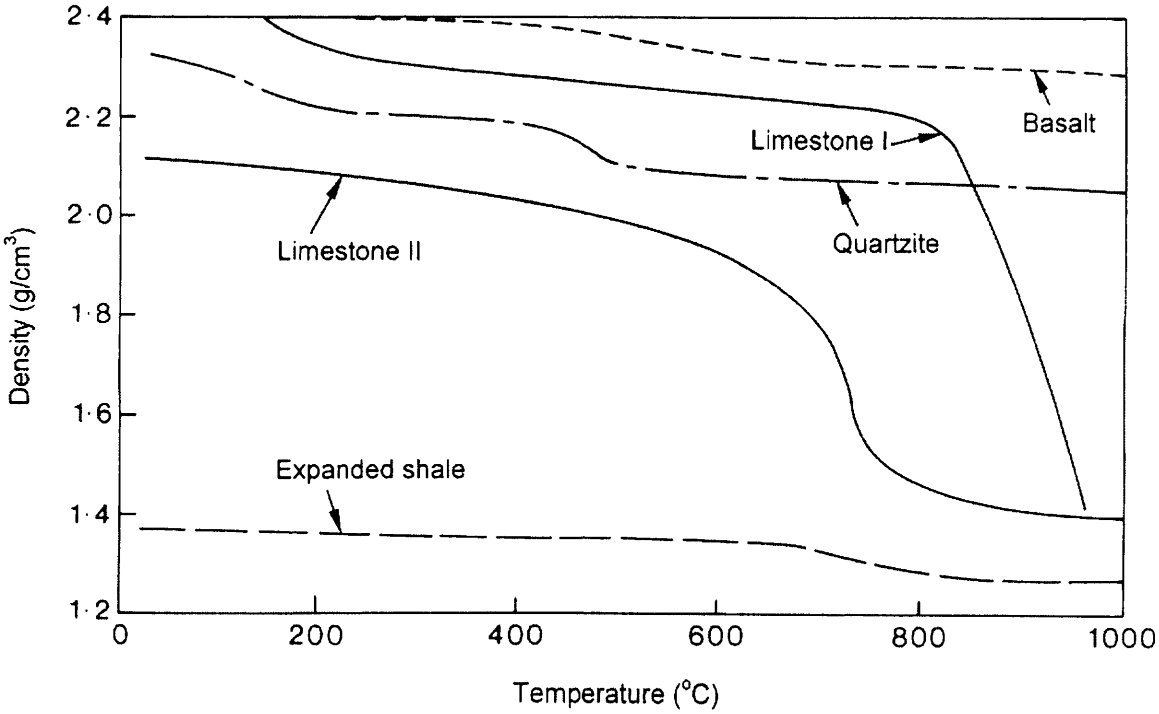


Figure 4.5: Density of concretes with four different aggregates against temperature. [HAR 73, SCH 82]. Note: Limestone concrete results from two different authors.

4.4 Specific heat

The specific which represents the heat capacity of concrete per units mass and temperature is also defined in terms of the enthalpy H at constant pressure P by :

$$Cp = \left(\partial H / \partial P\right)p \tag{eq 4.3}$$

It follows that if the enthalpy curve is available as a function of temperature, the specific heat can be estimated from the slope at any given temperature. Evaluation of the specific heat capacity is substantially complicated by the time dependent endothermic transformations that take place in concrete on heating. The 'sensible' portion of the specific heat can be measured by the drop calorimetry technique outlined in most elementary physics textbooks. The specific heat of concrete can also be evaluated from the component specific heats using an additivity theorem.

The specific heat value is sensitive to the various transformations that take place in concrete at elevated temperatures. These include the vapourisation of free water at about 100oC, the dissociation of $Ca(OH)_2$ into CaO and H_2O at about 400°C-500°C and the α-β quartz transformation in some aggregates. These time dependent changes are accompanied by the absorption of latent heat and are indicated by an apparent transient increase in specific heat. Specific heat is sometimes separated into "sensible" heat and "latent" heat based on the degree of physico-chemical transformations. The specific heat capacity of concrete is not only little affected by the mineralogical character of the aggregate but is also not very sensitive to the proportion of aggregate since the specific heat of aggregates are similar to that of the cement paste. The latent heat effects in concrete are, furthermore, less important owing to the presence of the aggregate whose stability is normally higher than that of the cement paste. The specific heat of concrete, however, increases considerably with an increase in moisture content due to the high specific heat of water (4.29 kJ/kg°C). A typical value for saturated concrete at 20°C is 1.0 kJ/kg°C. The value, however, may vary between 0.7 and 1.5 kJ/kg°C [BRO 68].

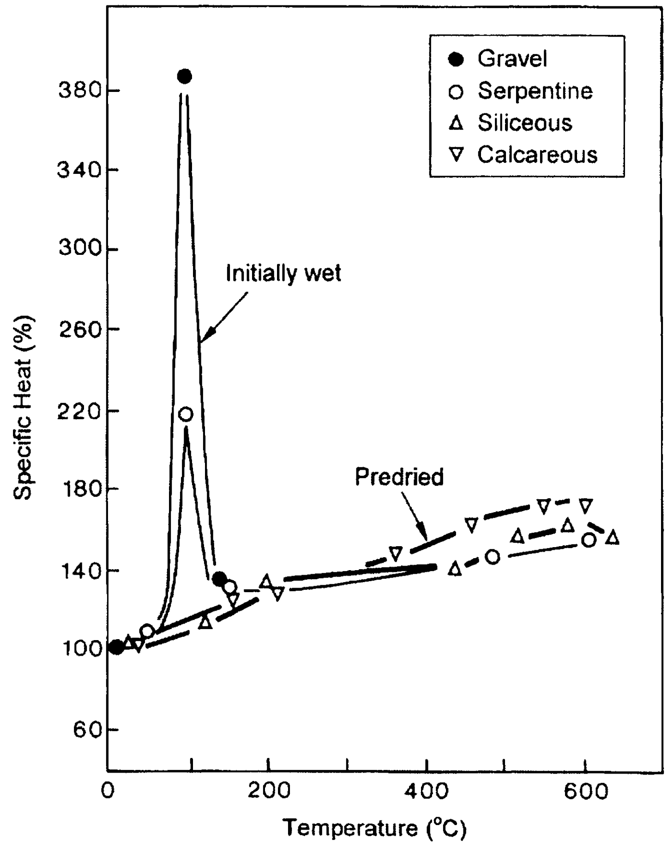


Figure 4.5: Effect of temperature on the specific heat of concrete excluding latent heat effects above 200°C compiled from four sources [IDI 75], [OHI 72], [HAR 67], [BLU 76].

Heating initially saturated concrete causes a rapid but temporary rise in specific heat at approximately 90°C of up to 380% [IDI 75] due to the rapid release of free water from the concrete and the absorption of latent heat of vaporisation (Figure 4.5). Heating permanently saturated (sealed) concrete apparently leads to similar variations in specific heat as in pre-dried concrete [BLU 76] but the dramatic increase at 90°C would not be expected to occur.

5 Strains during heating

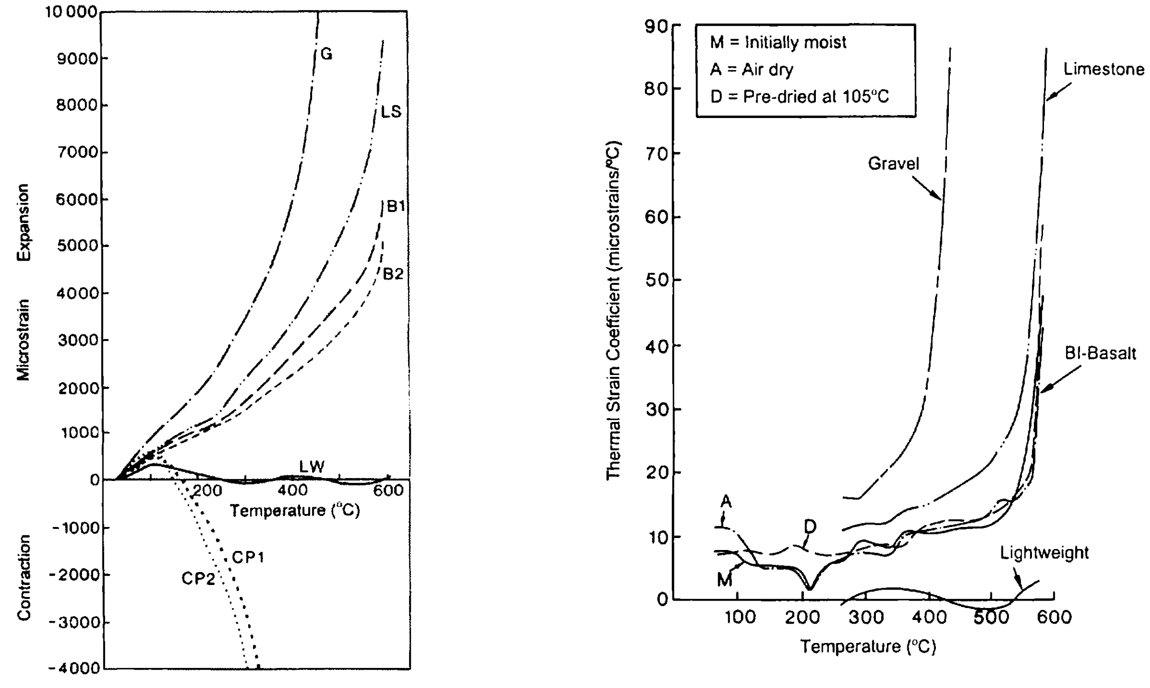
The thermal strains that develop during first heating-up under load can be considered to consist of "load-free" and "load-induced" components which have different and distinct properties.

5.1 Thermal strains

The *Thermal Strain* (TS) is strictly the strain of non-drying concrete measured when concrete is heated without applied load (i.e. load-free). It does not contain drying shrinkage.

In practice, for drying concrete, TS and drying shrinkage are determined together and are normally considered not to be separable. For this reason, the "total" strain of a drying unloaded concrete specimen measured during first heating is normally loosely called "thermal strain" in the literature. However, a method for isolating the shrinkage component that develops during first heating has been presented in [KHO 06c].

The thermal strain of drying concrete is a non-linear function of temperature, dominated by the aggregate type and content (Figure 5.1).



(a) Thermal strains (b) Coefficient of thermal strains

Figure 5.1: (a) Thermal strains and (b) Coefficient of thermal strains five concretes during first heating to 600^{o}C at 1^{o}C/minute. L = Limestone, BI & BII = Basalt, LW = Ligtweight, G = Gravel concretes. CPI&CPII = Cement Pastes [KHO 06a]. Note: BI and BII have the same course aggregate type and content but different cements (BI = OPC/PFA, BII, SRPC) and different sands.

For the concretes tested, thermal strain is more expansive for limestone than basalt concretes on account of the larger thermal expansion of limestone aggregate. Concretes containing quartz (in the sand or course aggregate) expand markedly at about 573^{o}C due to the α to β inversion of quartz. The effect of type of cement on the thermal strain of concrete, although smaller than the effect of the aggregate, is not negligible as can be deduced from Figure 5.1a (compare TS of BI & BII concretes). A minimum in the thermal strain coefficient occurs at 150-220°C which corresponds to the peak in the rate of moisture loss Figure 5.1b for BI concrete). The temperature at which this minimum occurs would depend upon the heating rate and the dimensions/geometry of the specimen [KHO 06a]. A summary of the influences of material and environmental factors upon the thermal strain of concrete is given in Table 5.1 indicating first order, second order and negligible influences.

Influence	**Factor**
First Order	• Temperature level • Type of the aggregate • Aggregate content by volume
Second Order	• Initial moisture content in the 65-85% RH range$^{\$}$ • Type of cement blend (aggregate content 65-75%) • Rate of heating in the range 0.2-1°C/minute
Negligible	• Age beyond three months after casting

$ While the initial moisture content has a significant effect on the thermal expansion of unsealed concrete at ambient temperatures, the influence is diminished at high temperatures and disappears altogether above 250°C

Table 5.1: Order of influence of material and environmental factors upon the thermal strain of concrete during first heating [KHO 06a].

5.2 Load Induced Thermal Strain (LITS)

The *Load-Induced Thermal Strain* (LITS) is obtained from the difference in strain between the thermal strain of an unloaded concrete specimen and the strain measured under constant load applied prior to heating and maintained constant during heating Figure 5.2, and after subtracting the initial elastic strain. The concrete mix, specimen, initial moisture content, moisture boundary condition (e.g. unsealed) and thermal boundary conditions (e.g. a constant rate of temperature increase of 1°C/minute) being the same in both tests. The only difference is the constant load level applied as a percentage of the compressive strength prior to heating (e.g. 0% and 2%).

LITS is relatively insensitive to the aggregate type and cement blend used because it originates in a common gel or C-S-H structure. LITS is similar for the three concretes up to 450°C providing the aggregate content, stress/cold strength ratio and pre-conditioning are similar (Figure 5.3). A common "master" LITS curve is therefore taken to exist for the three concretes for temperatures up to about 450°C. The presence of the master curve suggests that, for a given strength, the capacities of different concretes to relax stresses are similar and that the thermal stresses developing in heated structures would still be proportional to the expansion of the constituent aggregate even after relaxation. However, although LITS is not greatly influenced by aggregate type, it is (like the thermal strain) strongly dependent upon the aggregate content by volume and is restrained by it.

LITS is a non-linear function of temperature. A minimum in the LITS coefficient occurs at 110-180°C (Fig. 5.4) which corresponds to the peak in the rate of moisture loss [KHO 06a].

LITS is predominantly linearly related to stress in the load range up to 30% of the initial strength, and possibly higher [KHO 85b]. Generally, the following influences are small if not negligible: Concrete age (2 months to 9 years [KHO 87]); initial moisture content (65 to 100% R.H. [KHO 85b]); and type of concrete (e.g. master curve). These, together with the approximate linearity with stress, and the small influence of heating rate (e.g. for 0.5-1.0°C/minute [KHO 85b]), allow for considerable simplification of the analysis of heated concrete structures.

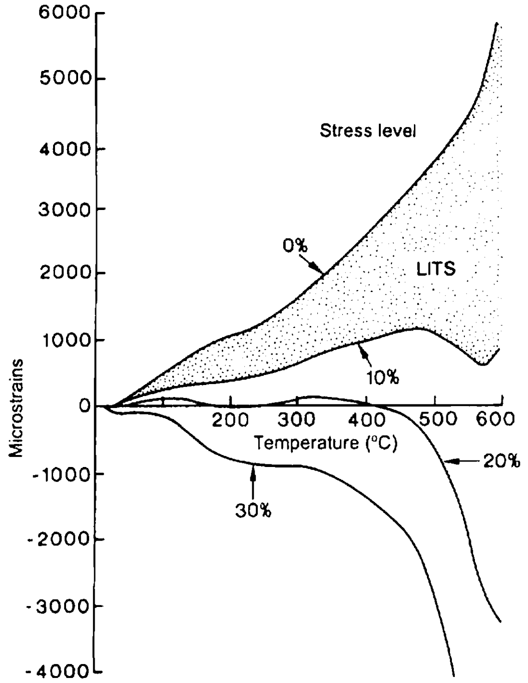


Figure 5.2: *Load Induced Thermal Strain (LITS) for the BI-Basalt concrete during heating at 1°C/minute, determined as the difference between the thermal strains determined under 0% and 10% load. Strains under 20% and 30% load also shown [KHO 06a].*

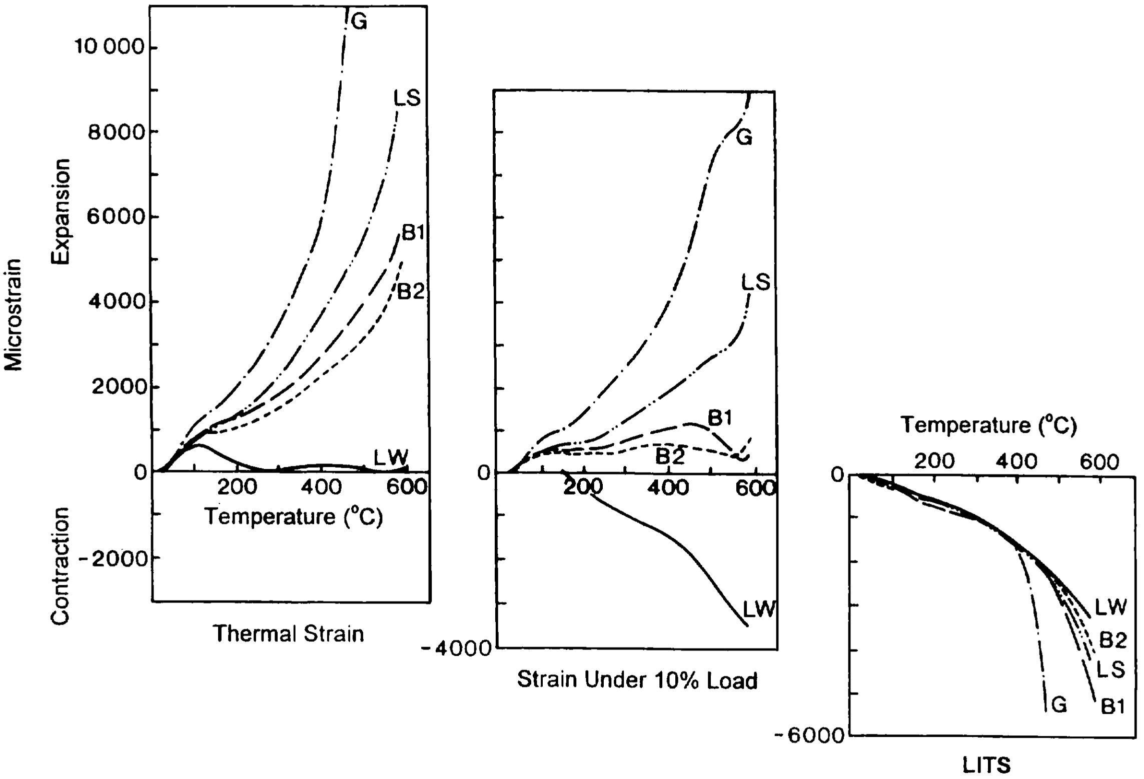


Figure 5.3: *Thermal strains of 5 concretes measured during first heating (from the initially air dry condition) at 1°C/minute under 0% (TS) and 10% load. The "Master" LITS (similar to the 5 concretes up to about 450°C) is determined as the contraction strain measured between the loaded and the unloded specimens.*
Note. The load-free Thermal Strain (TS) is dominated by the aggregate but not LITS which is seated in the cement paste [KHO 06a].

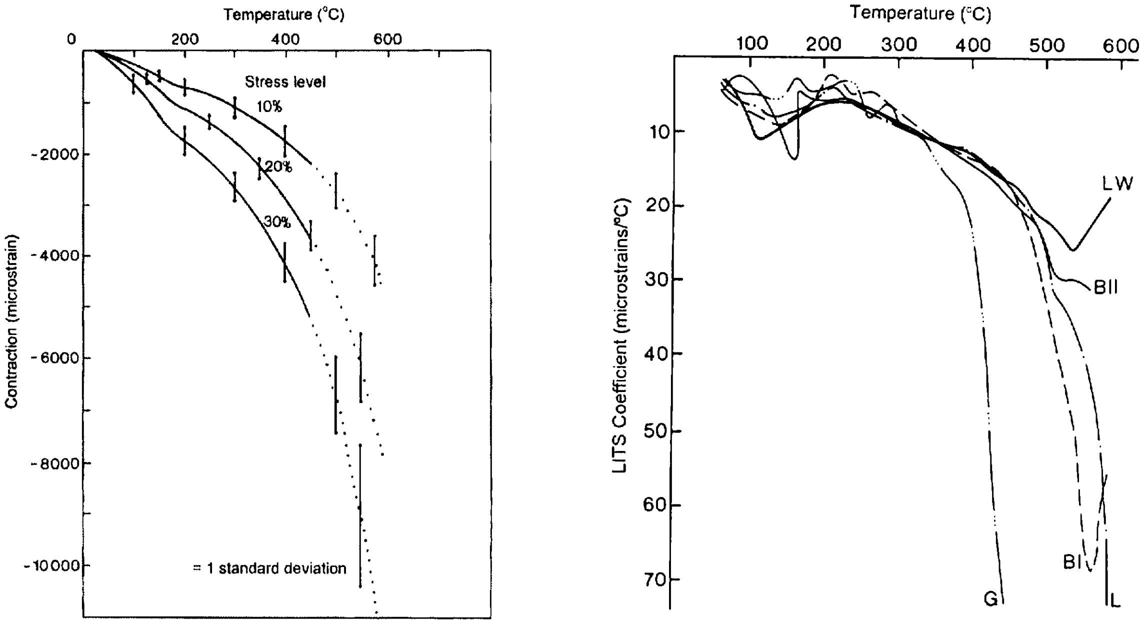
A summary of the influences of material and environmental factors upon the load-induced thermal strain of concrete is given in Table 5.2 indicating first order, second order and negligible influences.

LITS comprises several components including *transient strain/creep* (comprising *transitional thermal creep and drying creep), time-dependent creep,* and *changes in elastic strain* that occur during heating-up under load. LITS does not include the initial elastic strain on loading.

Transitional thermal creep (ttc) is strictly the strain that occurs in non-drying concrete induced by first time temperature increase under load [ILL 73]. Below 100°C, ttc develops over time to reach a limiting value at a period of up to about one month after heating under load. Transient strain/creep is designated for drying concrete and includes ttc and a drying creep component.

INFLUENCE	**FACTOR**
First Order	• Stress/strength ratio • Temperature • Aggregate content by volume (in the range 0-75%)
Second Order	• Rate of heating (in the range 0.2-1°C/minute) • Aggregate content by volume (in the range 65-75%) • Initial moisture content in the 65-85% RH range • Type of cement blend up to 450°C • Type of aggregate up to 450°C – providing it is stable
Negligible	• Age beyond three months after casting

Table 5.2: Order of influence of material and environmental factors upon LITS of concrete at high temperatures [KHO 06a].



(a) LITS (b) LITS Coefficient

Figure 5.4: (a) Master LITS and (b) LITS Coefficient of 5 concretes - during heating at 1°C/minute - under 10%, 20% and 30% load to 600°C [KHO 06a].
L = Limestone, BI & BII = Basalt, LW = Ligtweight, G = Gravel concretes.

Strictly, both ttc and transient creep do not contain any changes in the plastic strain that take place during heating under load. They should also not contain purely time-dependent strain - the influence of which is noticed when a concrete specimen is heated-up at a very slow rate when compared to one heated-up at a faster rate. In practice, these components are not separated out, and the LITS of non-drying concrete is loosely called "transitional thermal creep" and for drying concrete it is loosely called "transient creep".

Transient creep is normally by far the largest component of LITS (Figure 5.5) in unsealed mature concrete. It is irrecoverable on cooling and/or unloading and occurs only during first heating-up under load to a given temperature. However, it may reappear if a long period elapses between first and second heating. It develops rapidly above 100°C and is assumed by fire researchers to be temperature-dependent and not time-dependent. In practical terms this assumption can be made in fire applications which last only hours.

Drying creep is also non-recoverable on cooling/unloading, and is time- and temperature-dependent only in so far as moisture loss is time- and temperature-dependent. Changes in the elastic strains of concrete heated under load are small throughout the temperature range up to 500°C and even higher (not the case for concrete heated without load because of crack development). The test results show that, for loaded concrete, the variations with temperature of the elastic strains in the order of 10-100 microstrains were small when compared with the values of 1,500-9,500 microstrains for the residual strains [KHO 06b]. As for the time-dependent creep component, it is also relatively small as evident from specimens heated at 1oC/minute and 0.2°C/minute [KHO 85b]. Although the difference in the time-scale between the two heating rates was five times, the difference in the LITS was only about 5-20%. Time-dependent creep is mainly irrecoverable on unloading, although it may contain a "delayed elastic" component.

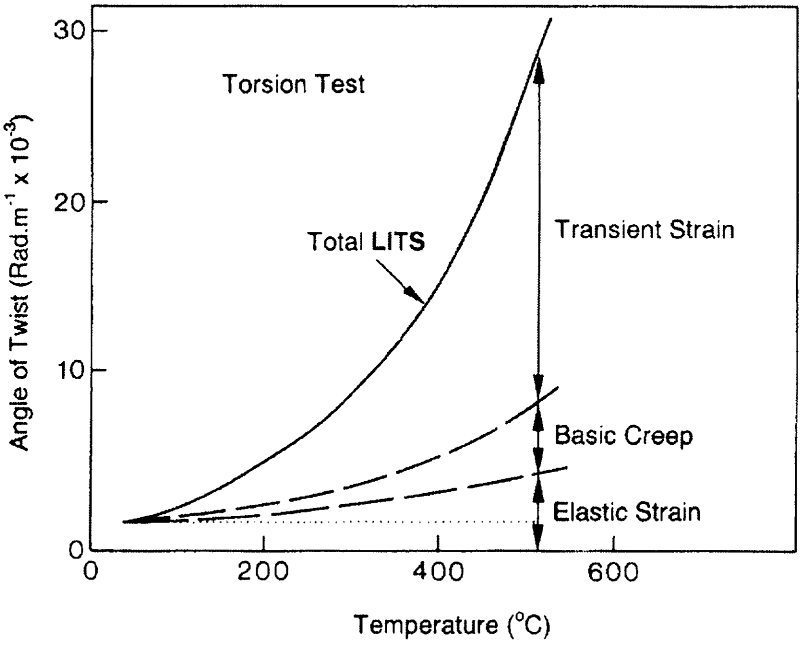


Figure 5.5: Development during first heating of LITS in torsion. Age at test 28 days [THE 74].

Note 1. In torsion testing, both TS and Shrinkage strains are not present.

Note 2. The elastic strain could change little with temperature when under 20% load [KHO 06a]

Considering the relative magnitudes of all the load-induced strain components mentioned above, it is evident that the bulk of LITS of drying concrete is largely non-recoverable on cooling and/or unloading. LITS is also largely temperature-dependent and not time-dependent, particularly above 110°C. It is for this reason that some authors prefer to call it transient "strain" rather than "creep". The test results explained below suggest the creep measurements taken for some time after constant test temperature has been attained were 'enhanced' due to internal physico-chemical instability of the concrete. The measured creep could, therefore, have contained at constant temperature a time-dependent "*delayed transient thermal creep*" component (including a drying creep component at lower temperatures). This may suggest that the term transient thermal *creep* would be more appropriate in nuclear applications than the term transient thermal *strain*, and the reverse is true for fire applications. The nature of this terminology has recently been the subject of some discussion. If it is taken as accepted that transient creep is indeed a time-dependent creep strain, then there is strictly no point in dividing it into two components. Nevertheless, since most authors feel that transient creep above 100°C occurs primarily during the thermal transient and is essentially temperature-dependent, then the use of the term delayed transient creep to denote the 'decaying' remains of transient creep which appear at constant temperature, as opposed to the part which develops during heating-up period, may become meaningful and helpful. It should however be noted that, for service conditions at constant temperatures, the duration of the "delayed transient creep" component could be small when compared with the total duration at temperature – possibly lasting for years in nuclear reactors. Structural engineers may, in this case, regard transient creep, including the delayed component, in their long-term analysis to be simply a function of temperature and not of time.

5.3 Strain components

5.3.1 Unloaded unsealed concrete

5.3.1.1 Thermal strain

The 'total' strain of unsealed concrete obtained during first heating without load ($\sigma = 0$) can be described in terms of three components [KHO 06c]:

$$\varepsilon^{T,0,d}_{tr,tot} = \varepsilon^{T,0,d}_{tr,th} + \varepsilon^{T,0,d}_{tr,sh} + \varepsilon^{T,0,d}_{tr,exp-crack} \qquad \text{(eq 5.1)}$$

The *thermal strain* $\varepsilon^{T,0,d}_{tr,th}$ is strictly the strain of non-drying concrete measured when concrete is heated without applied load. It does not contain drying shrinkage or a crack induced expansive strain generated by aggregate thermal instability or inter-particle tensile stresses. In practice, for drying concrete, the thermal strain ($\varepsilon^{T,0,d}_{tr,th}$) and the shrinkage ($\varepsilon^{T,0,d}_{tr,sh}$) strains are determined together and are normally considered not to be separable ($\varepsilon^{T,0,d}_{tr,th-sh}$). Together, they are simply called "thermal strain" in the literature. Care should be taken not to equate the true thermal strain of "unsealed" drying concrete with the thermal strain of "sealed" non-drying concrete particularly at temperatures above about 80°C. Therefore, only the strain components pertaining to unsealed concrete are presented. For this reason, the total strain of drying concrete is normally loosely called "thermal strain" in the literature. However, the two components of "true" thermal strain $\varepsilon^{T,0,d}_{tr,th}$ and shrinkage $\varepsilon^{T,0,d}_{tr,sh}$ can be uncoupled and treated separately. It should, therefore be emphasised that the thermal strain referred to here is the "true" thermal stain $\varepsilon^{T,0,d}_{tr,th}$ independent of shrinkage [KHO 06c].

$$\text{Traditional "thermal strain"} = \varepsilon^{T,0,d}_{tr,th-sh} = \varepsilon^{T,0,d}_{tr,th} + \varepsilon^{T,0,d}_{tr,sh} \qquad \text{(eq 5.2)}$$

It should be noted that the shrinkage strains at the lower temperature levels arise from moisture loss but at the higher temperature levels they are also caused by chemical dissociations (e.g. C-S-H, CH, $CaCO_3$).

5.3.1.2 Shrinkage

During first heating of virgin hardened and relatively mature concrete, the *shrinkage* component $\varepsilon_{\mathrm{tr,\,sh}}^{\mathrm{0,\,T,\,d}}$ would include "drying" shrinkage caused by (a) loss of physically bound water ($\varepsilon_{\mathrm{tr,\,sh\text{-}d}}^{\mathrm{0,T,\,d}}$) which takes place first, then (b) the loss of chemically bound water (dehydration of the cement paste and calcium hydroxide), and thereafter (c) De-carbonation (also chemical). The rate of loss of physically bound water depends upon the initial moisture content, the heating rate and the geometry and size of the specimen (e.g. surface area to volume ratio). The rate of chemical dissociations depend upon the heating rate and temperature level. Therefore [KHO 06c]:

$$\varepsilon_{\mathrm{tr,\,sh}}^{\mathrm{T,\,0,\,d}} = \varepsilon_{\mathrm{tr,\,sh\text{-}d}}^{\mathrm{T,\,0,\,d}} + \varepsilon_{\mathrm{tr,\,sh\text{-}chem}}^{\mathrm{T,\,0,\,d}} \qquad \text{(eq 5.3)}$$

5.3.1.3 Crack-induced strain

The sources of the *crack-induced expansive strain* $\varepsilon_{\mathrm{tr,\,exp\text{-}crack}}^{\mathrm{T,\,0,\,d}}$ [KHO 06c] could include excessive tensile thermal strains, the cracking of the aggregate which imposes stresses upon the matrix, and interface 'parasitic' cracking induced by differential thermal expansion/shrinkage between the aggregate and the paste. In fact, work published in the literature has hitherto normally ignored this component and also bundled it (along with drying shrinkage) as part of 'thermal strain'.

5.3.2 Loaded unsealed concrete

5.3.2.1 Total strain

The total strain of unsealed concrete obtained during first heating under load σ can be described in terms of six components [KHO 06c]:

$$\varepsilon_{\mathrm{tr,\,tot}}^{\mathrm{T},\sigma,\mathrm{d}} = \varepsilon_{\mathrm{co,\,el\text{-}pl,\,i}}^{\mathrm{T},\sigma,\mathrm{d}} + \varepsilon_{\mathrm{tr,th}}^{\mathrm{T,\,0,\,d}} + \varepsilon_{\mathrm{tr,sh}}^{\mathrm{T,\,0,\,d}} + \varepsilon_{\mathrm{tr,\,lits}}^{\mathrm{T},\sigma,\mathrm{d}} + \varepsilon_{\mathrm{tr,\,crack}}^{\mathrm{T},\sigma,\mathrm{d}} \qquad \text{(eq 5.4)}$$

where

$$\varepsilon_{\mathrm{co,\,el\text{-}pl,\,i}}^{\mathrm{T},\sigma,\mathrm{d}} = \varepsilon_{\mathrm{co,el,i}}^{\mathrm{T},\sigma,\mathrm{d}} + \varepsilon_{\mathrm{co,\,pl,\,i}}^{\mathrm{T},\sigma,\mathrm{d}} \qquad \text{(eq 5.5)}$$

is the "initial" elasto-plastic strain, uncoupled in equation 5, but determined experimentally together as in equation 4 during loading prior to heating, and is absent if the concrete is heated without load.

5.3.2.2 Load Induced Thermal Strain

The *Load-Induced Thermal Strain* (LITS) of drying unsealed concrete determined during first heating from the virgin state from two tests – one on a specimen heated without load and the other on an identical specimen heated under load, everything else being equal but excluding the initial elasto-plastic strain. Therefore, is determined experimentally as follows [KHO 06c]:

$$\text{LITS determined} = \varepsilon_{\mathrm{tr,\,lits}}^{\mathrm{T},\sigma,\mathrm{d}} = \varepsilon_{\mathrm{tr,tot}}^{\mathrm{T},\sigma,\mathrm{d}} - \varepsilon_{\mathrm{tr,tot}\cdot}^{\mathrm{T,\,0,\,d}} - \varepsilon_{\mathrm{co,el\text{-}pl,\,i}}^{\mathrm{T},\sigma,\mathrm{d}} \qquad \text{(eq 5.6)}$$

However, in the experimental procedure the specimen was initially load cycled to allow the initial elastic strains ($\varepsilon_{\mathrm{co,el,i}}^{\mathrm{T},\sigma,\mathrm{d}}$) to be determined.

LITS comprises four strain components:

$$\text{LITS components} = \varepsilon_{\mathrm{tr,\,lits}}^{\mathrm{T},\sigma,\mathrm{d}} = \varepsilon_{\mathrm{tr,\,ttc}}^{\mathrm{T,\,0,\,d}} + \varepsilon_{\mathrm{tr,\,cr\text{-}d}}^{\mathrm{T,\,0,\,d}} + \varepsilon_{\mathrm{tr,\,cr\text{-}t\text{-}dep}}^{\mathrm{T,\,0,\,d}} + \varepsilon_{\mathrm{tr,\,\Delta el}}^{\mathrm{T},\sigma,\mathrm{d}} \qquad \text{(eq 5.7)}$$

5.3.2.3 Transient creep/strain

The first two components of equation 5.7 are usually called transient creep (or transient strain) in the literature, and are erroneously confused as being LITS. It should be emphasised again that LITS comprises several components only one of which is transient strain/creep, albeit at high temperature. It is the largest of the components. Some literature mention Transient Creep or strain when what is meant is LITS because it is LITS that has been determined. Essentially, *transient creep* is a component of LITS of drying concrete, to distinguish it from *transitional thermal creep* of non-drying concrete [KHO 06a]. For unsealed concrete, transient creep comprises two components [KHO 06c]:

$$\text{Transient creep (or strain)} = \varepsilon_{\text{tr, cr}}^{\text{T, 0, d}} = \varepsilon_{\text{tr, ttc}}^{\text{T, 0, d}} + \varepsilon_{\text{tr, cr-d}}^{\text{T, 0, d}} \qquad \text{(eq 5.8)}$$

5.3.2.4 Time-dependent strain under transient loaded condition

Referring back to equation 5.7, the third term ($\varepsilon_{\text{tr, cr-t-dep}}^{\text{T, 0, d}}$) is a time-dependent strain noticeable when heating loaded concrete at two different rates. The smaller the heating rate the larger will this component be. Basic creep is essentially the strain of thermally stabilised concrete when loaded at constant temperature. Strictly, therefore, the use of the term "basic" creep in this context is wrong by definition. The author prefers to just call it "time-dependent" strain that develops under transient heating conditions under load. It may even be a "delayed" form of LITS. For this reason, LITS should be related to a specific heating rate. In this section it is 1°C/minute.

LITS is largely irrecoverable on cooling and/or unloading [KHO 85b].

5.4 Isolation of shrinkage and creep components

The method adopted in this section has essentially two purposes [KHO 06c]:

- isolate and quantify individual components and assess their magnitude and significance;
- determine the suitability of employing the isolated components in a predictive model.

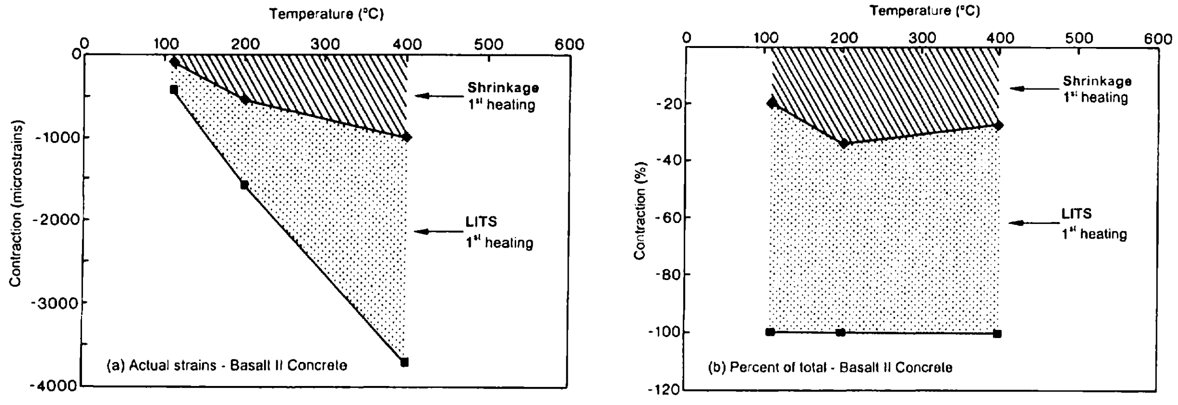
A number of the strain components (e.g. shrinkage at constant temperature $\varepsilon_{\text{co, sh}, t_1 - t_2}^{0, \text{T, d}}$) can be determined directly from the "total" measured deformation and, therefore, require only one test (i.e. one specimen). Other strain components (e.g. LITS or $\varepsilon_{\text{tr, lits}}^{\sigma, \text{T, d}}$) require two tests (or 2 specimens) for each stress level (one specimen heated without load and another heated under load - everything else being equal. The strains that could be determined directly from individual specimens during the first heat cycle are listed in Table 5.3.

Stage - Thermal Cycle	**Unloaded Specimen (S^0)**	**Loaded Specimen (S^σ)**
First heating-up	$\varepsilon_{\text{tr, tot}}^{\text{T, 0, d}}$	$\varepsilon_{\text{tr, tot}}^{\text{T}, \sigma, \text{d}}$
Constant temperature	$\varepsilon_{\text{co, tot, 1h-5}}^{\text{T, 0, d}}$ $(= \varepsilon_{\text{co, sh, 1h-5}}^{\text{T, 0, d}})$	$\varepsilon_{\text{co, tot, 1h-5}}^{\text{T}, \sigma, \text{d}}$
Residual	$\varepsilon_{\text{co, res, 1c}}^{20, 0, \text{d}}$	$\varepsilon_{\text{co, res, 1c}}^{20, \sigma, \text{d}}$

Table 5.3: Strain components determined directly from individual specimens during the first heat cycle [KHO 06a].

5.4.1 Drying shrinkage

The shrinkage component ($\varepsilon_{\text{tr, sh}}^{0, \text{T, d}}$) during first heating-up to a test temperature of 'T' is evaluated indirectly from the difference between residual shrinkage strains measured after first cooling and the shrinkage measured at temperature for the same unloaded specimen. This is valid in the absence of crack-induced and chemically-induced expansion. Figure 5.6 gives both the shrinkage and LITS components that have been separated out by this method.



(a) actual strains (b) percent of total

Figure 5.6. Shrinkage and LITS components of the Basalt II concrete during first heating at 1°C/minute under a load of 20% of the initial unheated compressive strength and in the absence of cracking [KHO 06c].

The accumulated drying shrinkage obtained during heating-up to 110°C was a small proportion (0-13%) of the total residual shrinkage. In other words, the bulk of drying shrinkage has taken place after the test temperature of 110°C has been reached. This is for a cylindrical specimen 60mm in diameter and 180mm long. Size effects then need to be considered for different sized specimens. This conclusion is compatible with the moisture loss measurements which show that the 60mm diameter Basalt II specimens heated to 105°C at 1°C/minute would have lost only 3% of the ultimate water loss for that temperature.

The shrinkage component developing during heating increases steeply with increasing test temperature in the range 100-200°C to reach about 60% of the residual shrinkage. The actual shrinkage during heating increases further at 400°C but the percentage of the residual shrinkage does not change significantly from the 60%, perhaps because the shrinkage related to the dissociation of the calcium hydroxide develops only slowly above 400°C.

5.4.2 LITS Component

The LITS component ($\varepsilon^{\sigma,T,d}_{tr,\,lits}$) is always a contraction and non-recoverable upon cooling [KHO 06c]. It develops during first heating-up to a test temperature of 'T' and is evaluated indirectly from the difference between the 'total' strain measured during first heating-up of the loaded and unloaded specimens. Again, in the absence of cracking (valid below the critical temperature):

$\varepsilon^{T,\sigma,d}_{tr,\,lits} = \varepsilon^{T,\sigma,\bar{d}}_{tr,tot}\,.\,\varepsilon^{T,0,d}_{tr,tot}$ [2 specimens required S^0 & S^σ] (eq 5.9)

The bulk of the contraction of the loaded specimens occurring during heating-up to 100°C consists of LITS (Figure 5.6). The shrinkage component being small (i.e. 0-20% of the total). The shrinkage component increases to about 30-35% of the total contraction during heating-up to 200°C but reduces again to about 20-27% at 400°C (Figure 5.6b). Therefore, throughout the temperature range 20-400°C, LITS constitutes in excess of 65% of the total contraction during first heating.

6 Spalling of concrete

6.1 Definition, significance and types of spalling

Spalling is the violent or non-violent breaking off of layers or pieces of concrete from the surface of a structural element when it is exposed to high and rapidly rising temperatures as experienced in fires. [KHO 07], [KHO 06e]

Almost the entire body of knowledge of the spalling of concrete comes from experience of buildings and other concrete structures exposed in fire. New information now emerges from spalling in tunnels which experiences more severe temperature-time curves and where the initial moisture content of the concrete (e.g. 75%) is normally higher than that in buildings (55%). The rates of heating of the concrete surface in building fires are typically 20-30°C and in tunnel fires can rise by about 250°C/minute.

Spalling could be grouped into six categories:

- aggregate spalling,
- explosive spalling
- surface spalling,
- sloughing off spalling,
- corner spalling and
- post cooling spalling.

It is also possible to categorise these types of spalling differently as no clear dividing line exists between some of them. In a single fire several of those types of spalling could occur, even all of them. The first three normally occur early into a fire while the last three normally occur later in a fire. Surface and Explosive Spalling are violent while Corner Spalling, sloughing off is non-violent.

The extent, severity and nature of occurrence of spalling is varied. Spalling may be insignificant in amount and consequence, such as when surface pitting occurs. Alternatively, it can have a serious effect on the fire resistance of the structural element because of extensive removal of concrete which exposes the core of the section, and the reinforcing steel or tendons, to a more rapid rise in temperature, thus reducing the load-bearing cross-sectional area.

The unexpected damage caused to a concrete construction by spalling can render fire safety design calculations inaccurate and lead to significantly reduced levels of safety for concrete structures in the event of fire. The inability to predict the occurrence of spalling has been a limiting factor in the development of robust models of the response of concrete structures to fire.

6.2 Explosive spalling

Explosive spalling is considered to be the most serious form of spalling in a fire situation. It could result in the explosive removal of a concrete layer up to 25-100mm thick. Multiple spalling could occur in high strength concrete that contains silica fume. Such multiple spalling occurred recently in the Channel Tunnel between the UK and France where the concrete strength was in excess of 100 MPa. Many factors have been identified as influencing explosive spalling. These include concrete strength, age and permeability, heating rate and profile, section size and shape, type and size of aggregate, moisture content, the presence of cracks, reinforcement, polypropylene fibres and imposed loading. The influence of the different parameters is given in Table 6.1. There are two forms of explosive spalling, both influenced by external loading; pore pressure spalling and thermal stress spalling [KHO 06e]. They act singly or on combination depending upon the section size, the material, and the moisture content. Multiple explosive spalling has been experienced in "high performance concretes" using silica fume, taking a layer of material at a time during a fire.

Spalling	Porbabilistic Time of Occurrence	Nature	Sound	Influence	Main Influences
Aggregate	7-30 mins	Splitting	Popping	Superficial	H, A, S, D, W
Corner	30-90 mins	Non-violent	None	Can be serious	T, A, Ft, R
Surface	7-30 mins	Violent	Cracking	Can be serious	H, W, P, Ft
Explosive	7-30 mins	Violent	Loud bang	Serious	H, A, S, Fs, G, L, O, P Q, R, S, W, Z
Sloughing-off	When concrete weakens	Non-violent	None	Can be serious	T, Fs, L, Q, R
Post-cooling	During & after cooling upon absorption of moistur	Non-Violent	None	Can be serious	As sloughing off but also W1, AT

A = Aggregate thermal expansion
D = Aggregate thermal diffusivity
Fs = Shear strength of concrete
Ft = Tensile strength of concrete
G = Age of concrete
H = Heating Rate
L = Loading/restraint
O = Heating profile
AT = Aggregate type
P = Permeability
Q = Section shape
R = Reinforcement
S = Aggregate size
T = Max temperature
W = Moisture content
Z = Section size
W1 = Moisture absorption

Table 6.1: Characteristics of the different forms of spalling [KHO 07], [KHO 02c]

6.2.1 Factors influencing explosive spalling

6.2.1.1 Permeability

Permeability is an important factor influencing the critical pressure level, because it affects the rate of vapour release [KHO 07] [SER 77]. It has been suggested that spalling is unlikely to occur if the permeability of the concrete is greater than about 5x10^{-11} cm^2 [HAR 65]. Concrete of higher quality generally possesses higher density and a finer pore structure and therefore offers higher resistance to moisture flow.

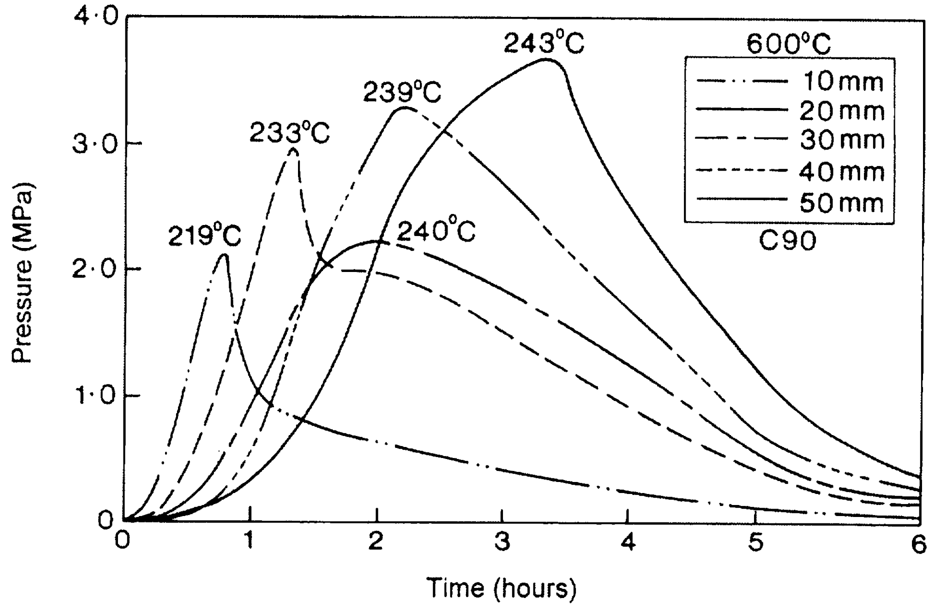


Figure 6.1: Pressure distribution for concrete slab exposed to a radiative heat of 600°C from one side [KAL 97].

6.2.1.2 Age of concrete

The influence of age of a concrete on its susceptibility to spalling has been subject of conflicting reports. The majority of reports, however, suggest that the probability of spalling reduces with age, although this may be due to the lower moisture level in older concretes.

6.2.1.3 Strength of concrete

Ironically, poor quality concrete is superior to good concrete in spalling. Concrete that is classed as "*high performance*" at room temperature - because of its high strength, low permeability and consequently good durability - is in fact a "low performance" concrete at high temperature because of the increased susceptibility to spalling [HER 84], [SAN 93], [JUM 89]. Higher strength is achieved by reducing the water/cement ratio. In recent years, this was augmented by the use of silica-fume which produced a dense concrete of very low permeability. Silica-fume concrete has a high susceptibility to explosive spalling even at low heating rates. However, in general, reducing the w/c ratio would enhance pore pressure spalling (via lower permeability) but reduce thermal stress spalling (via higher strength).

6.2.1.4 Compressive stress and restraint

Applied loads, and restraint, increase the susceptibility of concrete members to spalling [JUM 89], [MEY 72], [MEY 77]. The 1982 revision of the FIP/CEB recommendations [FIP 78] suggest that compressive stresses should be limited. An increase in compressive stress, either by reduction in section size or an increase in loading, encourages explosive spalling. The initial compressive stress in the exposed layer of concrete may not by itself promote spalling. However, high compressive stresses - caused by restraint to thermal expansion - develop when the rate of heating is such that the stresses cannot be relieved by creep quickly enough. Combinations of compressive stresses (above 2 N/mm^2) and moisture contents (above 3.3% by weight) make the occurrence of spalling likely in a fire.

6.2.1.5 Type of aggregate

Although the test results from different authors are inconsistent, it can generally be concluded that the likelihood of thermal stress explosive spalling is less for concrete containing a low thermal expansion aggregate. The risk of explosive spalling increased in the following order: Lightweight, basalt, limestone, siliceous, Thames River gravel. However, this only applies for concrete with relatively dry aggregates, since it has been shown that lightweight aggregate concrete has a high susceptibility to spalling if the aggregate is saturated.

6.2.1.6 Aggregate size

The available evidence suggests that increasing the aggregate size promoted explosive spalling [MAL 84], [CON 95].

6.2.1.7 Cracking

Internal cracking has a dual and opposite effect upon explosive spalling. While microcracks facilitate the escape of moisture during heating and thus relieve pore pressures, they also facilitate the process of spalling by providing sites for crack propagation.

6.2.1.8 Reinforcement

Tests results suggest that spalling is less likely with the provision of reinforcement in the central region a slab, and occurred at a higher heating rate [MEY 72]. The presence or absence of reinforcement was found to be a more important factor in spalling than the quantity of reinforcement [ICI 75]. However, congestion of steel bars or tendons, with only small spaces between them is considered to induce the formation of cracks, and may therefore promote spalling.

6.2.1.9 Cover to reinforcement

If the cover exceeds 40 mm for dense or 50 mm for lightweight aggregates concrete, there is a danger of concrete spalling [BS 85]. Concrete cover thicknesses of 15 mm or less, seem less prone to serious spalling, probably because the mass of unsupported concrete is not large [CON 95].

6.2.1.10 Supplementary reinforcement

The use of a light mesh cover does not prevent spalling but could limit the extent of spalling and significantly improve the performance of the columns with regards to fire resistance [CEM 79]. Supplementary reinforcement protected concrete columns from the effects of spalling, but not from the phenomenon itself. Control of spalling by use of supplementary reinforcement is sometimes used in sections where the cover to the outer bars exceeds 40 mm. However, supplementary reinforcement is difficult to place in thin sections, such as ribbed floors. Supplementary reinforcement provides two benefits: Limiting fire damage and easier repair of the structure. It is recommended that such a mesh be used only in cases where high standards of fire resistance have to be met [COP 79], [COP 80].

6.2.1.11 Steel fibres

The addition of a steel fibre reinforcement did not eliminate the explosions in very dense silica-fume concrete cylinders subject to heating at a rate of 1°C per minute. In fact the increase in tensile strength produced a more violent explosion because of the sudden release of a greater amount of energy.

6.2.1.12 Polypropylene fibres

Recently, it was found that the addition of 0.05-0.1% by weight of polypropylene fibres in a concrete mix eliminated explosive spalling even in high strength concrete (60-110 N/mm^2) but not necessarily in ultra high strength concrete (>150 N/mm^2). The science and technology of the use of non-metallic polymer fibres to combat explosive spalling of concrete is currently being developed in the current multi-national Eureka project NewCon [KHO 06d], [KHO 06e], experimentally (large and small-scale including microstructure) and by numerical modelling. The project aims to separate the different types of spalling.

6.2.1.13 Air-entrainment

The use of air-entraining agent could remove the risk of explosive spalling [HER 84]. The addition of the agent had the effect of reducing the moisture content and increasing the absorption value. In effect, it reduced the pore saturation thus alleviating pore pressures. The summary given here will be described in full in reference [KHO 07].

6.2.2 Mechanisms of explosive spalling

A detailed examination of all the theories and mechanisms contributing to spalling in fire conditions is beyond the scope of this work [KHO 07]. A summary of the main features is given in this Chapter.

Each form of spalling (aggregate, surface, explosive, corner, pot-cooling, sloughing-off) has its own mechanism for occurrence. Sometimes, mechanisms overlap between the different forms. For the sake of brevity, we discuss here only two main mechanisms of **explosive** spalling singly and combined.

6.2.2.1 Pore pressure spalling

Pore pressure spalling has been predicted using a "moisture clog model" [SHO 65], "vapour drag forces model" [MEY 72] or an "idealised spherical pore model" [AKH 73]. The main factors which influence pore pressure spalling are the permeability of the concrete, the initial water saturation level, and the rate of heating. Pore pressure spalling may apply by itself only for small unloaded specimens. For larger specimens, the pore pressure will have to be considered together with both the thermal and load stresses before the likelihood of explosive spalling can be assessed.

6.2.2.2 Thermal stress spalling

Thermal stress spalling: At sufficiently high heating rates, ceramics and dry concrete can experience explosive spalling. This is attributed to excessive thermal stresses generated by rapid heating and demonstrates that factors other than pore pressures may contribute to explosive spalling. Heating concrete generates pressure gradients (Figure 6.2) that induce compressive stresses close to the heated surface (due to restrained thermal expansion) and tensile stresses in the cooler interior regions. Surface compression may be augmented by load or prestress, which are super-imposed upon the thermal stresses. However, very few concrete structures are loaded to levels where the necessary failure stress state is reached. This makes thermal stress spalling - by itself - a relatively rare (but not impossible) occurrence.

6.2.2.3 Combined thermal stress and pore pressure spalling

Combined thermal stress and pore pressures: explosive spalling generally occurs under the combined action of pore pressure, compression in the exposed surface region, and internal cracking (Figure 6.2). Cracks develop parallel to the surface when the sum of the stresses exceed the tensile strength of the material. This is accompanied by a sudden release of energy and a violent failure of the heated surface region

The most effective methods to reduce the risk of explosive spalling include: (a) the use of a thermal barrier, (b) employing polypropylene fibres, (c) employing an air-entraining agent, (d) use of low thermal expansion aggregate. In addition, appropriate use of reinforcement could limit the extent of spalling, though not prevent the phenomenon itself. The risk of spalling is also reduced if the moisture content is low and the permeability of the concrete is high.

6.3 Methods of analysis used

Despite decades of research and testing, the prediction of spalling is still not a fully developed science. Important advances have been made recently in the understanding of the fundamental underlying mechanisms that influence spalling. Until now, the prediction of spalling during heating has been largely an imprecise empirical exercise. The methods attempted could be classified as follows:

6.3.1 Large scale tests

Large-scale tests are commonly used to assess the spalling of tunnel elements conducted for a given fire scenario. Normally 2-3 specimens are involved. If two specimens do not spall that the element passes the test. However, this is a statistically low number since in some tests

10 specimens were tested with 5 spalling and 5 non-spalling. Nevertheless this is the most reliable method but very costly.

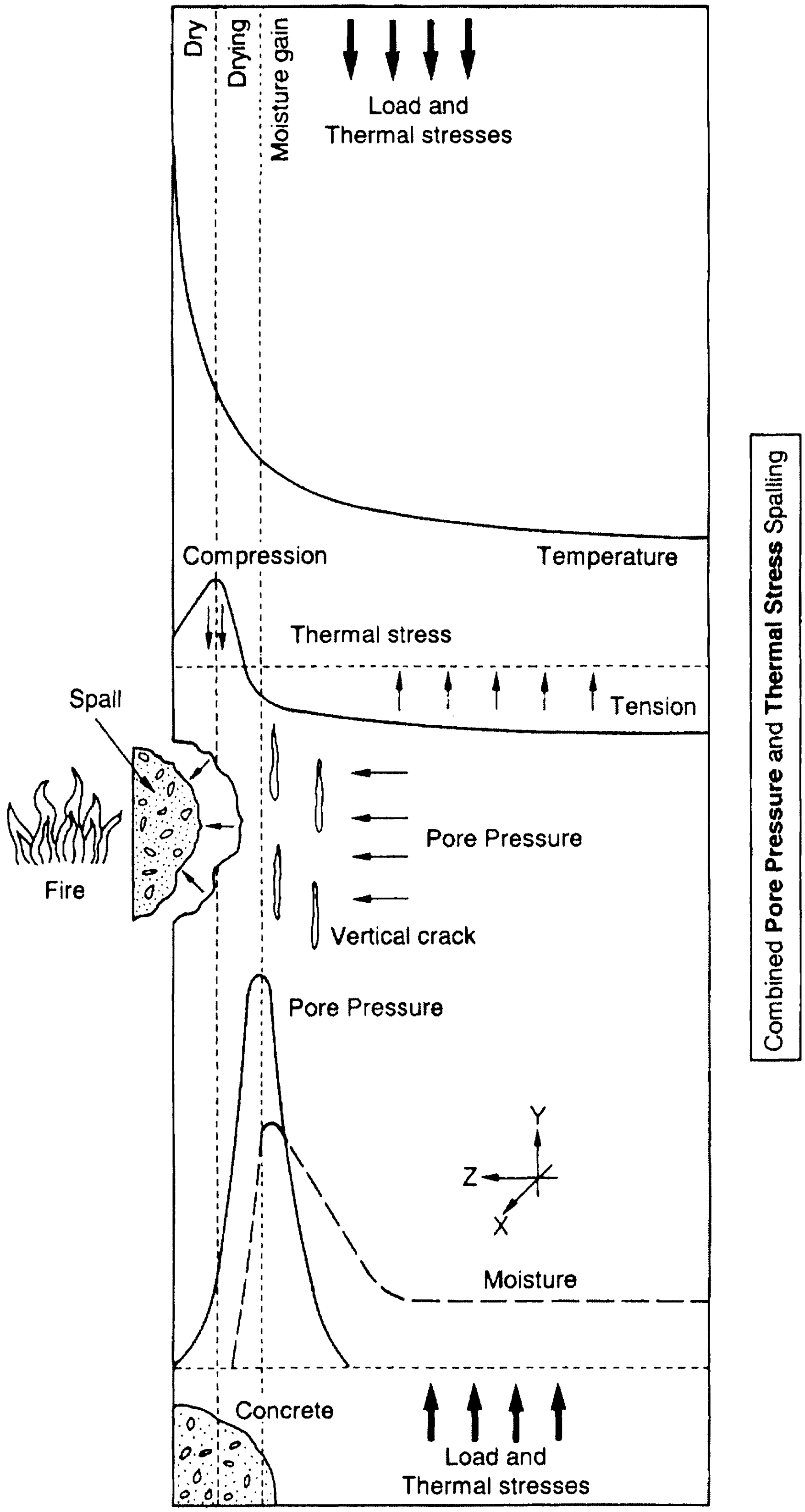


Figure 6.2: Explosive spalling as influenced by pore pressures and thermal stresses [KHO 07].

6.3.2 Use of nomograms

Previous testing in research institutions have produced nomograms indicating influences of various parameters (Figures 6.3 & 6.4). These nomograms should not be taken literally and could be misleading, but are helpful to indicated influences. These nomograms show only two influencing parameters or three but no more. Influences are shown but key elements are not indicated such as: Type of aggregate, size of aggregate, concrete strength/permeability, additives, reinforcement etc. Importantly, these previous tests were conducted on a material and under fire conditions which may differ greatly from the case being investigated. Also importantly the nomograms do not show the stochastic nature of concrete. In reality there exists no dividing line between spalling and no spalling zones.

6.3.3 Theoretical models

Theoretical modelling of the different forms of spalling has been attempted in the past with no major success. The complex combined nature of the influences of moisture content, pore pressures and thermal stresses in the heterogeneous concrete material - with complex pore structure which varies markedly with temperature during first heating - do not lend themselves easily to analytical modelling [CON 95]. Theoretical models have failed to provide predictions of spalling to any acceptable level, and to date, cannot not be usefully employed.

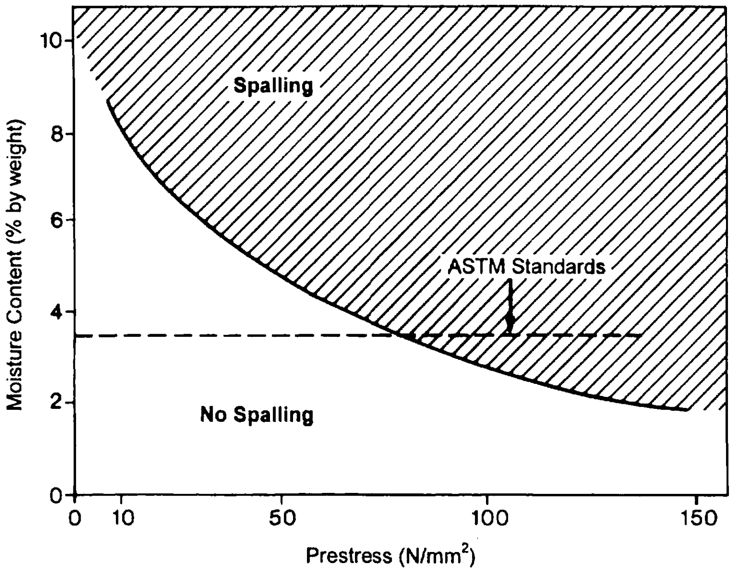


Figure 6.3: Explosive spalling nomogram showing only two influencing parameters and no stochastic trends (after Christiaanse [CHR 72]).

Note. Nomograms should not be taken literally but only as indications of trends. In reality there exists no dividing line between spalling and no spalling zones. There should be a "dividing" zone as spalling is a stochastic phenomenon.

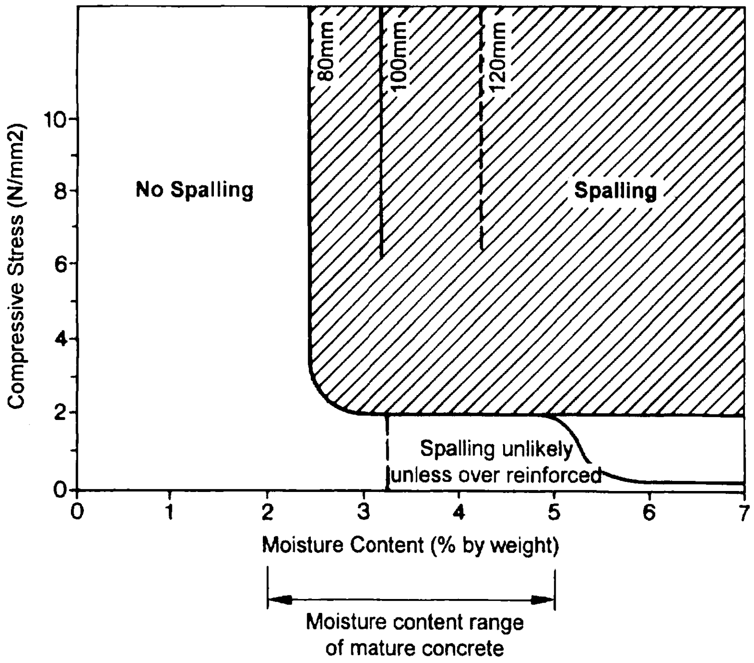


Figure 6.4: Explosive spalling results of Meyer-Ottens [MEY 72] showing three influencing parameters and here again not stochastic trends.

6.3.4 Numerical models

In the view of the author numerical methods are the most likely to succeed, employing fully coupled thermo-hydro-mechanical models. These models are being developed but are not yet ready for use.

6.3.5 Expert assessment

This method is used in the absence of the other methods mentioned above. The example given below falls under this category.

6.4 Passive fire protection against spalling

Passive fire protection in tunnels is addressed by the judicial use of:

- ***thermal barriers*** which reduce the flow of heat to the concrete and steel reinforcement;
- ***polypropylene fibres*** in the concrete mix itself (precast, insitu or sprayed) to essentially reduce steam pore pressures, and hence the risk of explosive spalling. This is investigated in the NewCon project [KHO 06d].

While each structure is unique, concrete structures in general could be classified into various categories depending upon design and location. In terms of passive fire protection a key difference is whether they are

- ***existing structures*** requiring upgrading, carried out by the use of thermal barriers; or
- ***new structures*** in which fire safety could be an inherent part of the holistic cost-effective design from the outset, thus rendering possible the judicial employment of thermal barriers and polypropylene fibres.

However, it should be noted that, unlike thermal barriers, polypropylene fibres (which are of lower cost per unit surface area of concrete exposed to fire) do not reduce temperatures in any practical sense but operate in a complex manner within the concrete itself to reduce pore pressures.

6.5 Simple tunnel example of spalling assessment

This is the method that will be used here as a hypothetical example. The stochastic approach taken in this example is new and intended to be simply a rough guide. It is not comprehensive and the approach itself needs further development [KHO 07].

6.5.1 Influence of concrete parameters

In this hypothetical example, as a first step in a generic sense, the influence upon explosive spalling of the concrete parameters provided are given in Table 6.2 showing "generic" net effects, including intangible/qualitative evaluations. It uses transitive scale -3 to +3 and is based on the experience of the author. The -3 indicates the worst possible effect upon spalling and +3 indicates the best possible effect. Negative values indicate that the likelihood of spalling increased.

Concrete Parameter	Value	Impact
Moisture content	4% (75% RH)	-3
Max Aggregate size	32mm	-1
Aggregate type	Granite	+2
Concrete strength[1]	28-29 MPa	+2
Water/Cement Ratio[1]	At least 0.45-0.50	+2
Applied load[2]	0%	0
Applied load[2]	60%	-2
ISO 824 fire[3]	Applied load 0%	-1
RABT 30 Fire[3]	Applied load 60%	-2
Separating wall thickness	200-300mm	0

Note 1: The low strength is largely influenced by the high W/C producing higher than normal permeability
Note 2: Two scenarios of applied load with different impacts. Load as percent of unheated concrete strength.
Note 3: Two scenarios of fire scenario with different impacts

Table 6.2: Assessment of impact of concrete parameters provided [KHO 07].

Table 6.2 is tentative and is indicative of trends. It cannot be used for a reliable assessment, but for an indication of potential influences. We observe that the applied loads give different impacts and so do the fire scenarios. The probabilities given below in Table 6.3 are estimated on the basis of this information.

It should be noted that an initial applied load of 60% will significantly increase with heating as the concrete tries to expand but is restrained by the rest of the lining. This increases the risk of explosive spalling.

6.5.2 Explosive spalling assessment

The assessment of the likelihood of exploding spalling given in Table 6.2 draws on the assessment of the relative impacts of different parameters given in Table 6.3. The probabilities are given separately for the different fire scenarios, different mix of vehicles and the loading applied upon the wall and ceiling. Since the actual vehicle mix in the particular tunnel comprises 96% cars - the weighted average of likelihood for explosive spalling tends towards the low-medium likelihood where load is applied.

Load Condition	**Vehicle Mix**	**Likelihood of Explosive Spalling**			
		Low	Low-Med	Med-High	High
		10-20%	20-40%	40-55%	55-65%
0% Load	Cars	▨			
60% Load	Cars		▨		
0% Load	Cars & HGV		▨		
60% Load	Cars & HGV			▨	
0% Load	HGVs			▨	
60% Load	HGVs				▨

Table 6.3. Tentative explosive spalling assessment based on parameters provided regardless of traffic frequency [KHO 07].Load is the percent of the unheated concrete strength.

7 Mechanical properties

7.1 Stress-strain relation and mix design

Changes in mechanical properties of concrete upon heating may be attributed to "*material*" factors such as:

- Physico-chemical changes in the cement paste
- Physico-chemical changes in the aggregate
- Chemical interaction between the aggregate and cement paste
- Thermal incompatibility between the aggregate and the cement paste

and are influenced by "*environmental*" factors, such as:

- Temperature level
- Heating rate
- Applied loading
- External sealing influencing moisture loss from the surface

These changes are not always detrimental but in some situations can lead in enhanced strength depending on the temperature level, mix constituents, applied load and moisture condition.

7.2 Stress-strain relation

Compressive strength is by far the most tested property of heated concrete, because it is a vital element of structural design and usually gives an overall impression of the quality of the material. However, strength (i.e. ultimate stress) is only one of several properties derived from the stress-strain relation of concrete which include the modulus of elasticity, the strain at ultimate stress, the maximum strain and the mechanical energy dissipated. This section is, therefore, devoted to the treatment of the influence of temperature upon the full stress-strain relation in compression and in tension and the properties derived from it, for normal, high strength and ultra-high strength concretes as well as for sealed and unsealed concretes. It should be noted that the stress-strain relation is normally tested at constant temperature and the function of temperature is derived from tests performed at different levels of constant temperature. The derived properties, therefore, also relate to the behaviour of the material at constant elevated temperature. In this respect, the properties do not represent dynamic fire exposure where the structure thermo-hydro-mechanical state of the structure changes throughout the concrete cross-section.

In structural design of heated reinforced concrete, the entire stress-strain curve, often in idealised form, must be considered as a function of temperature. Ideally, the curves in both compression and tension should be considered, although the latter is not critical when, for example, modelling the total deformation to failure of building elements (beams and columns) exposed to fire. The actual stress-strain curve for concrete at elevated temperatures, as opposed to simply the modulus of elasticity, is important since in an accident situation (fire in ordinary structures) the concrete could be exposed to excessively high temperatures such that the inelastic range of the stress-strain curve is likely to be reached.

In compression, concrete is, to a certain degree, non-linearly elastic. In tests, particularly of concrete heated to the test temperature in an unstressed condition, a fairly significant concave-up part of the curve is normally encountered at the beginning of loading owing to the closing of pre-existing fine cracks. To counter this effect, and to achieve seating of the gauges, and to minimise creep, Rilem recommends a small "preload" of 0.05 MPa, followed by three load cycles to 15% of the reference strength, to be applied with rates of loading and unloading of 0.5 MPa/second prior to loading to failure at the same rate [RIL 95]. The curvature of the "elastic" range (in contrast with the more linear stress-strain relation of the individual cement paste and aggregate constituents) is attributed to the presence of interfaces between the cement paste and the aggregate and the development of bond microcracks at those interfaces.

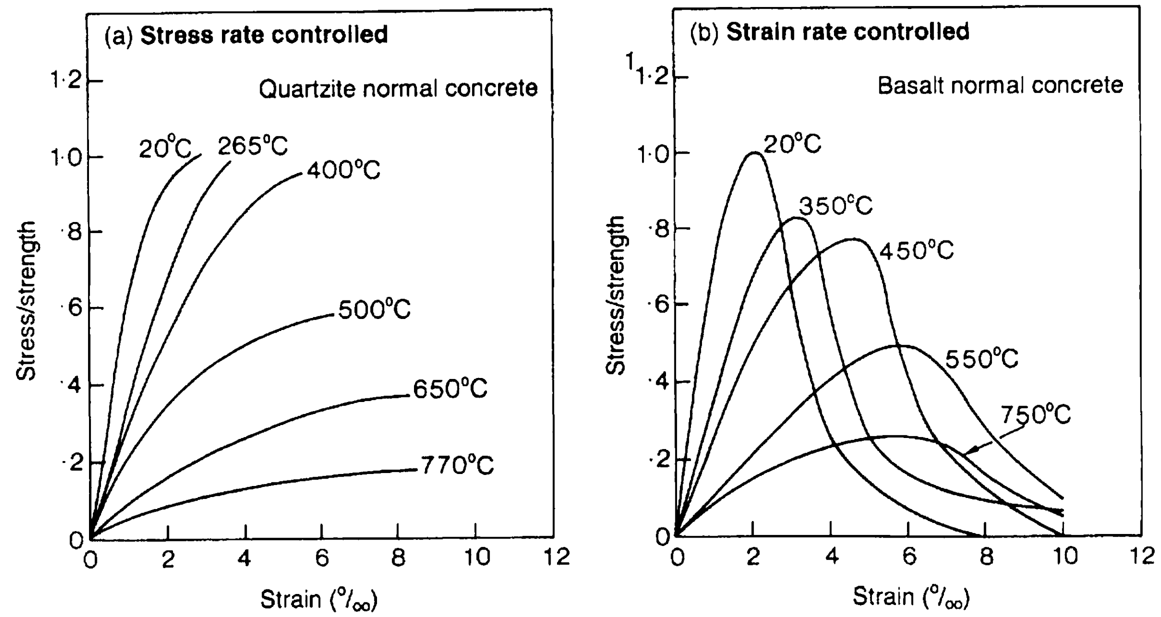


Figure 7.1: Effect of test method upon the stress-strain relation of concrete at temperatures from ambient up to about 750°C – (a) stress rate control [AND 73], (b) strain rate control [SCH 81].

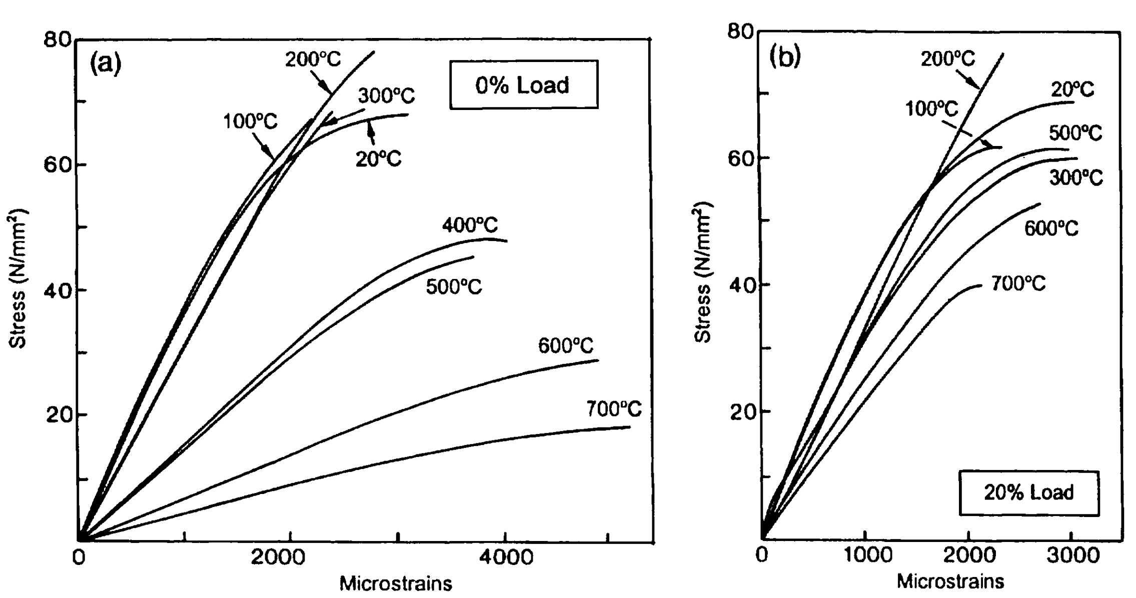


Figure 7.2: Effect of load level during heating-up upon the residual stress-strain relation in uniaxial compression of unsealed C70 HITECO concrete cylindrical specimens [KHO 99]. Note: The tests were conducted at constant stress rate so the descending branch is not shown. Load level is defined as percent of the unheated concrete strength.

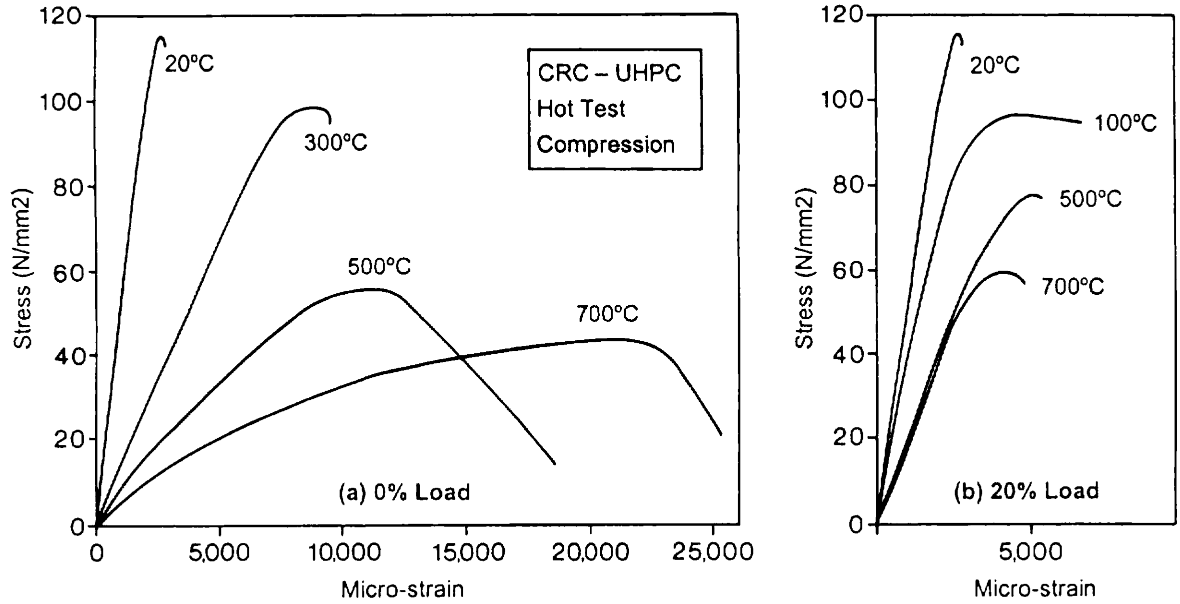


Figure 7.3: Effect of load level during heating-up upon the stress-strain relation at temperature in uniaxial compression of unsealed Ultra High Strength (UHSC) concrete cylindrical specimens containing polypropylene and steel fibres [KHO 99]. Test carried out under deformation control.

For concrete heated without load, an increase in the test temperature results in a reduction in the slope of the elastic range (i.e. reduced modulus of elasticity), a reduction in the maximum stress (i.e. reduced compressive strength) and also a reduction in the slope of the descending branch (i.e. less brittle and more ductile material). In contrast, the ultimate strain at maximum stress, and crushing strain, increase with temperature (Figures 7.1-7.3).

However, for concrete heated under compressive load the effect of temperature is markedly less such that a significant part of the ascending branch can be very similar for a range of temperatures as far apart as 20°C and 500°C (Figure 7.2-7.3). Such a dramatic influence of pre-loading is not normally appreciated and quite often modellers use the stress-strain results of concrete tested after heating without load. The compressive stress could mitigate against the development of microcracks and could also cause a "densification" of the paste reflected phenomenologically by a large Load Induced Thermal Strain and micro-structurally by an increase in silicate polymerization. Interestingly the load effect has been observed in normal strength concrete, high strength concrete "HSC" and ultra high strength concrete "UHSC" containing steel fibres (Figure7.2-7.3). The influence of pre-load was also found for concrete tested "hot" and also, but possibly to a lesser degree, for concrete tested after cooling - providing the cooling process also takes place under load. Cooling without load could result in a very marked increase in the elastic strain [KHO 86b]. However, the effect of load level during heating per se is not consistent from the literature. Some results indicate a progressive "benefit" with increase in load level (e.g. 0%, 10% and 30% as measured for a gravel concrete heated to 250°C and 450°C - but others indicate no discernible difference for pre-load levels of 17%, 33% and 50% - as in the case of a quartz concrete heated to 600°C. The influence of loading during heating is, therefore, a complex phenomenon dependent upon the type of concrete and the heating/loading conditions, but what seems to be consistent is a general "beneficial" effect of pre-loading.

For concrete heated in the sealed condition without moisture loss (e.g. inner zones) , the slope of the stress-strain curve can change significantly with time as "hydrothermal" chemical and physical transformation take place over time above 100°C. A progressive reduction in the slope of the stress-strain curve has been observed over time at 150°C of sealed limestone concrete, and during thermal cycling to 150°C, due to physico-chemical changes [BER 72].

Measurement of the stress-strain relation in direct tension at high temperature requires a special testing machine and method of analysis. At room temperature, the stress-strain curve in tension appears to be similar in shape to that in compression. In direct tension, the development of cracks has the effect both of reducing the effective area resisting stress and of increasing the contribution of cracks to the overall strain. This may be the reason why the departure from linearity of the stress-strain relation in tension occurs at a slightly lower stress/strength ratio than in compression [GUO 87].

The actual stress-strain curve for concrete at elevated temperatures, as opposed to simply the modulus of elasticity, is important since in an accident situation (fire in order structures or a nuclear reactor accident) the concrete could be exposed to excessively high temperatures such that the inelastic range of the stress-strain curve is likely to be reached.

7.2.1 Maximum temperature level

The maximum temperature level to which the concrete is heated has a major influence upon the stress-strain relation whether tested "hot" at temperature or "residual" after cooling. However, the influence of temperature varies significantly depending upon the loading and moisture conditions experienced by the material as explained below.

7.2.2 Load level during heating

A key influence upon the stress-strain relation of heated concrete which is not usually appreciated is that of the load level during heating. Concrete heated under load exhibits a different behaviour from concrete heated without load. This influences not only the stress-strain relation as a function of temperature level but also the derived properties such as the strength and modulus of elasticity. As with thermal strain and creep, the loading-heating sequence is vitally important and is usually ignored by people carrying out the tests or by those employing test results in modelling and structural analysis. It is normal practice for cylindrical specimens to be cast (stress history is also ignored in practically all cases), and then at a given age the specimens are inserted in a furnace and heated without load to a given temperature and then loaded uniaxially to failure at temperature - or after cooling. Therefore, in most cases, load history and then loading during the heat cycle are not simulated. In presenting the results of stress-strain tests, or when using them, it is important to clearly note the load level during the heat cycle.

7.2.3 Hot or residual testing

Another issue is whether the tests were conducted "hot" at temperature or "residual" after cooling. In the absence of the Load Induced Thermal Strain (LITS) during the cooling process, thermal and inter-particle stresses and related cracking could develop thus influencing significantly the stress-strain relation and its derived properties. The thermal stresses during cooling would depend upon the rate of cooling and upon the thermal shock presented by any quenching that is applied. Also during cooling, the absorption of moisture from the atmosphere could have a significant influence upon the physico-chemical structure of the concrete and hence its stress-strain behaviour. This effect is a function of the maximum temperature level prior to cooling, the amount of drying (of evaporable water – e.g. physically bound water), dehydration (of non-evaporable water – e.g. calcium hydroxide and C-S-H), and dissociation (e.g. of calcium carbonate) taking place during the heat cycle. Re-hydration could result in "healing" as new hydration products are formed but also in "damage" upon the expansive rehydration of CaO.

7.2.4 Moisture condition

The moisture condition of the concrete during the heat cycle is important especially in nuclear reactor applications and in concrete exposed to fire where the heating rate and moisture condition result in the build-up of moisture in the concrete mass. Sealed moist concrete during heating undergoes increased hydration, chemical reactions between different products, and at temperatures above 100°C chemical transformations of the C-S-H that significantly influence its mechanical properties. However, the majority of tests are carried out on relatively small specimens in the "unsealed" condition.

7.2.5 Loading technique

As in the case with room temperature tests, the loading to failure of heated concrete can be performed by controlling the "stress rate" or the "strain rate". The latter producing a post peak descending branch representing strain softening of the concrete. Stress-strain test results of heated concrete are available in the literature from both types of tests but the majority have so far been from stress rate controlled tests (Figure 7.1)

7.3 Modulus of elasticity

Structural behaviour at high temperatures is often dependent upon the modulus of elasticity of concrete. Although, there are several methods of measuring the modulus of elasticity of concrete when there is no straight portion of the curve, the "*initial tangent*" modulus is of little practical importance, and the "*tangent*" modulus at any point is valid for only a very small portion of the curve. The "*secant*" modulus on the other hand satisfies the arbitrary distinction that deformation during loading is considered elastic, and any subsequent strain increase is regarded as creep. RILEM, therefore, recommends that the secant modulus of elasticity be determined for the stress range of between 15% and 30% of the reference strength at temperature "hot" or after cooling "residual". It is clear that the "*dynamic*" modulus of elasticity (as can be determined by an ultrasonic technique) is not a standard procedure for high temperature testing. It is supposed to refer to almost purely elastic effects and is little influenced by creep since only a negligible stress is applied. For this reason the dynamic modulus is appreciably higher than the static (secant) modulus and is approximately equal to the initial tangent modulus, although it should be appreciated that there does not exist a single relation between the two moduli based on physical behaviour [NEV 95]. The dynamic modulus of elasticity is sometimes regarded [BAZ 76] as a more meaningful measure of the temperature effect on the elastic response of concrete owing to its smaller creep component. Authors that measured the dynamic modulus of elasticity include Philleo [PHI 58], Crispino [CRI 72], Berwanger and Sarkar [BER 73] and Kishitani and Shiiba [KIS 76].

7.3.1 Modulus of elasticity and strength

The modulus of elasticity is frequently related to compressive strength and there is no doubt that the modulus of elasticity of concrete "E_c" increases with increase in the compressive strength of concrete, but there is no agreement on the precise form of the relationship at room temperature let alone at high temperatures. This is not surprising, given that E_c is affected by the modulus of elasticity of the aggregate "E_{agg}" and by its volumetric proportion in concrete. E_{agg} is normally higher than that of the cement paste but E_{agg} of lightweight aggregate differs little from that of hardened cement paste. At room temperature the increase in E_c is progressively lower than the increase in compressive strength.

Stronger concrete has a higher modulus of elasticity, but for any two concretes the ratio of 'E' values is considerably smaller than the ratio of strengths. Weaker concrete, furthermore, tends to experience a greater loss of modulus with increase in temperatures [KAP 72]. However, while this is generally true for normal strength concrete, the E value trends for higher strength concretes appear not to be related to strength (Figure 7.5) possibly due to the use of steel fibres, silica fume, heat pre-compression etc.

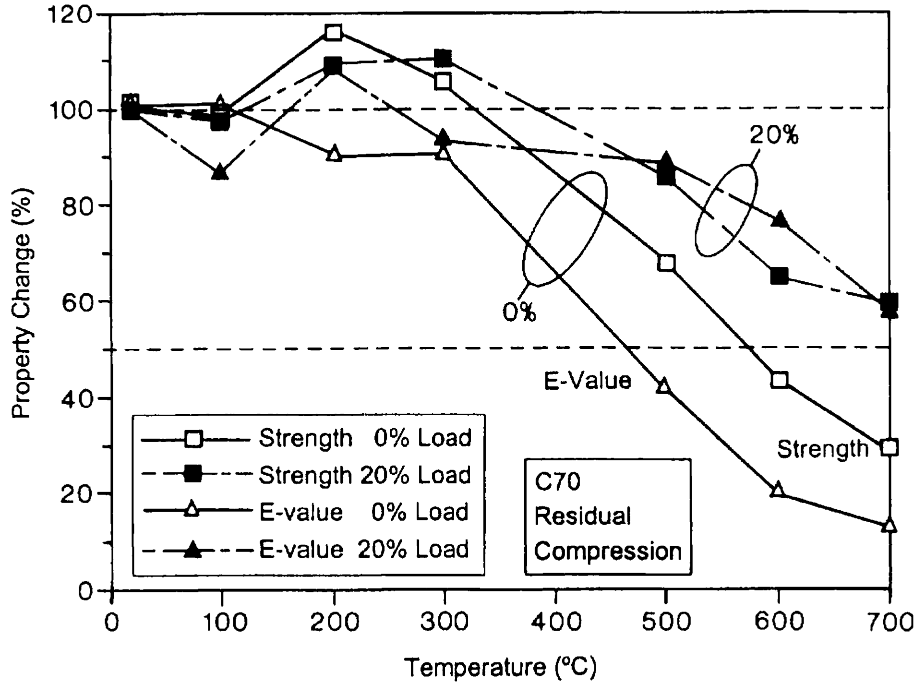


Figure 7.4: Relative reduction of residual strength and E value C70 concrete heat cycled without load and under 20% load. [KHO 99].

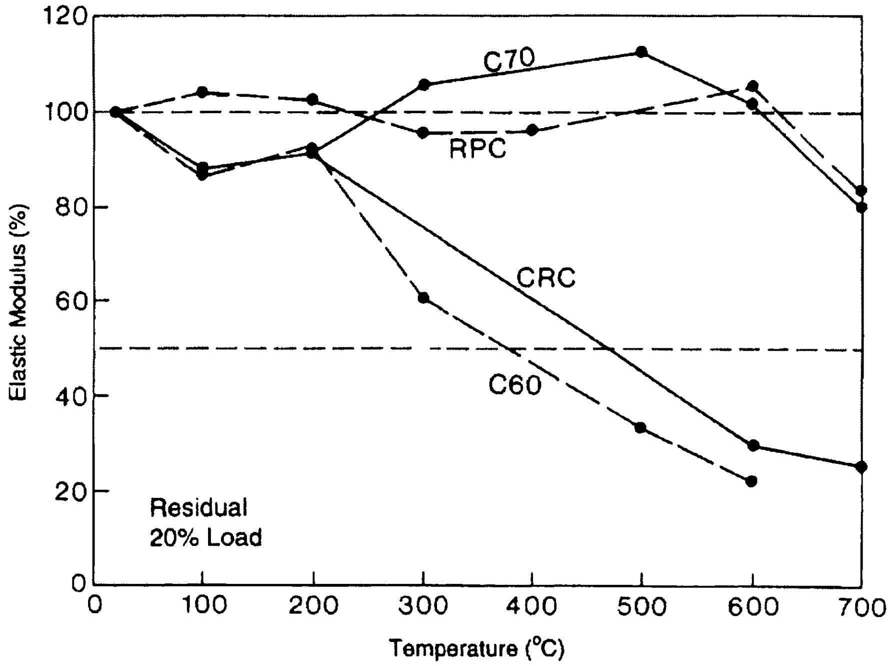


Figure 7.5: Relative Moduli of elasticity of two high strength concretes (C60 & C70) and two ultra-high strength concretes (CRC & RPC with steel fibres) measured after a heat cycle under 20% load indicating lack of influence of concrete strength [KHO 99].

The effect of temperature on the ratio Strength/Elastic modulus for unsealed concrete was determined by Marechal [MAR 72]. The departure from linearity for temperatures below

100°C is caused by the loss of evaporable water which tends to increase the strength but decrease the modulus of elasticity. Above 100°C temperature affects the elasticity and strength of some concretes similarly and therefore the ratio strength/E remains relatively invariant with increase in temperature [LAN 71, MAR 72, NEV 95]. This, for example, was found applicable for unsealed quartzite [MAR 72] and gravel [LAN 71] concretes. The trend with some other concretes is for the elasticity to experience a larger drop with increase in temperature than the strength [MAR 72]. Marechal relates the greater drop of elasticity for the siliceous limestone aggregate concrete to the destruction of bonds and the increase in plasticity with temperature. Lankard's results for sealed gravel concrete [LAN 71] indicate a drop in E value far greater than strength for temperatures up to 80°C, but a comparable fall for temperatures above 80°C. Nishizawa [NIS 72], however, found that E depends on compressive strength regardless of temperature (up to 90°C) or duration of heat exposure and concluded that the E value of reactor vessel concrete at elevated temperatures up to 90°C could be determined from the strength/E relationship at ambient temperatures. Because of the greater drop of E value than strength below 100°C for both sealed and unsealed concrete, the percentage drop in E value for temperatures above 100°C are usually greater than that for strength.

7.3.2 Temperature effect

Increase in temperature generally leads to a continuous fall in the modulus of elasticity of both unsealed and sealed concrete, the fall in the latter being the greater. The recorded elasticity values depend to a large extent on whether the concrete is loaded during heating and on whether the measurements are taken at test temperature or after cooling.

Results of reduction in E value with temperature taken from different tests indicate considerable variations up to about 80°C. Above 100°C, further reduction generally tends to be less steep and approximately linear; and appears to be similar for different types of concrete as shown in Figure 7.4. For unsealed Thames gravel concrete, the E curve reaches a minimum at 80°C (owing to moisture dilation) followed by a maximum at 100°C and a steady decline above this temperature.

The wide variation in the magnitude of the percentage reduction in the modulus of elasticity at 80°C of different concretes tested under similar conditions is most likely to be caused by the different aggregates used. Due to thermal incompatibility with the cement paste, concretes containing aggregates of low thermal expansion such as limestones or dolomites experience a greater reduction in the modulus of elasticity than those with a higher thermal expansion like gravels and sandstone.

Variation in results from different authors can be caused by a number of factors in addition to the type of aggregate used; these include effects of different water/cement ratio, definition of the modulus of elasticity and the degree with which the time-dependent creep strains are separated from the instantaneous strains. Other factors relating to test conditions include whether the specimen was tested 'hot' or after cooling, on whether the specimen was pre-loaded before testing and on the rate of heating and cooling as well as the rate of loading.

Above 100°C the fall in E value with increase in temperature seems to be approximately linear for all types of concrete up to a critical temperature range within which the concrete experiences severe deterioration which is accompanied by a sharper decrease in the modulus of elasticity for river gravel concrete above 400°C and expanded shale concrete above 600°C.

The more severe fall in E value for sealed concrete is attributed to the hydrothermal reactions that normally result in weaker hydrates [LAN 71]. The E value of sealed gravel concrete can fall to only 42% of its ambient value at 80°C.

The duration of exposure of unsealed concrete to elevated temperatures does not significantly affect the modulus of elasticity once moisture loss has ceased.

The E value of sealed concrete on the other hand falls with duration of exposure to temperature. Conversely the E value of sealed concrete can increase with time (up to certain temperatures only of about 120 to 150°C) if reactive silica is present in sufficient quantities.

7.3.3 Effect of pre-loading

The effect of pre-loading on the stress-strain relation of concrete at elevated temperatures is considerable as shown in Figures 7.2-7.4.

7.4 Compressive strength

Before explaining the changes in strength of concrete as a function of increasing temperature, two positive aspects of heated concrete need pointing out.

7.4.1 Influence of LITS

Up to the 1970s, scientists were puzzled as to why concrete does not break up when heated above 100°C, because the differential strain resulting from the expansion of the aggregate and shrinkage of the cement paste is far too large (Figure 7.8) to be accommodated by elastic strains. The discovery of the phenomenon of the Load Induced Thermal Strain (LITS) which includes a "transient creep/strain" component for concrete heated under compression provided the answer. LITS is much larger than the elastic strain, and contributes to a significant relaxation and redistribution of thermal stresses in heated concrete structures. This phenomenon is still not fully appreciated by structural engineers and should be incorporated more fully into standards and design codes. It should be noted that LITS appears in compression but not in tension [HAG 05].

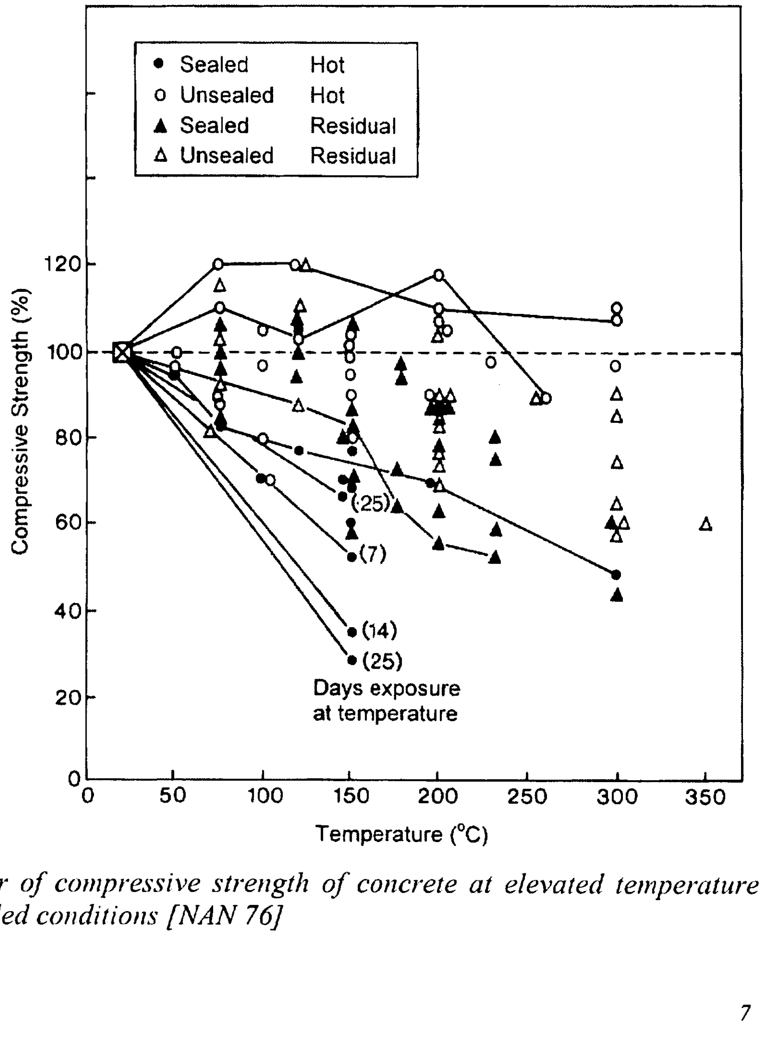


Figure 7.6: Scatter of compressive strength of concrete at elevated temperature under sealed and unsealed conditions [NAN 76]

7.4.2 Influence of loading during heating

Another positive aspect of concrete at high temperature is the "beneficial" influence of loading which places the material into compression and "compacts" the concrete during heating and inhibits the development of cracks. Again, this influence is not fully appreciated by the practicing engineer and is not adequately covered in the codes and standards. An example of this phenomenon is given in Figure 7.2-7.4 and illustratively in Figure 7.7. The influence of temperature can be markedly diminished when concrete is heated under load. Both the compressive strength and elastic modulus reduce far less with increase in temperature for the concrete heated under load. However, for concrete heated without load, tests consistently show the modulus of elasticity to reduce with increase in temperature by a larger proportion of the initial value than the compressive strength.

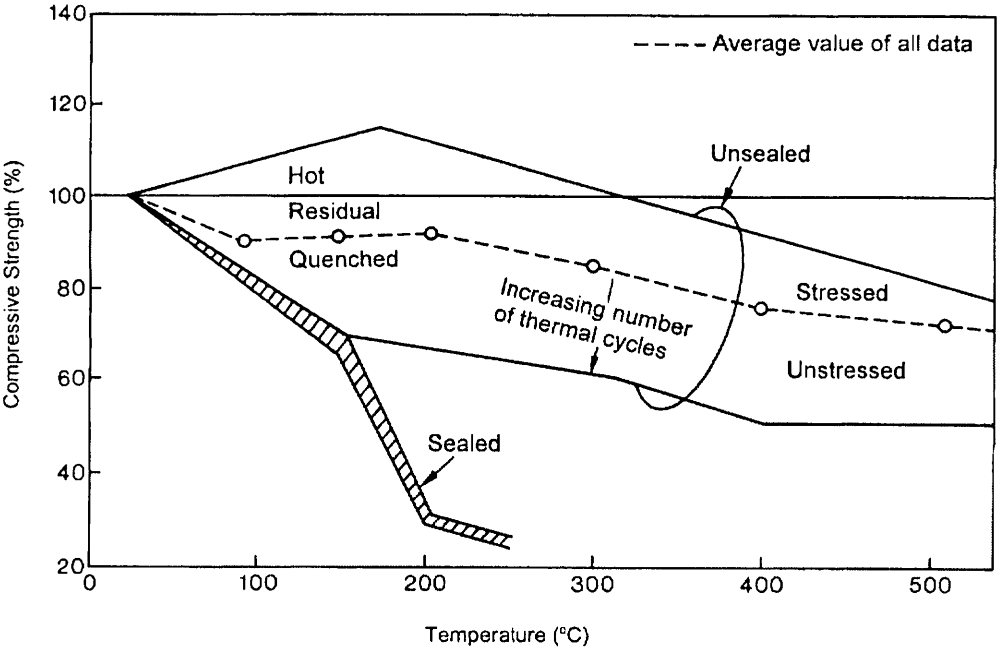


Figure 7.7: Explanation of some of the scatter in the strength of concrete at high temperature in terms of sealing condition, stress level during heating, testing hot or residual or thermal cycling. [LAN 71]

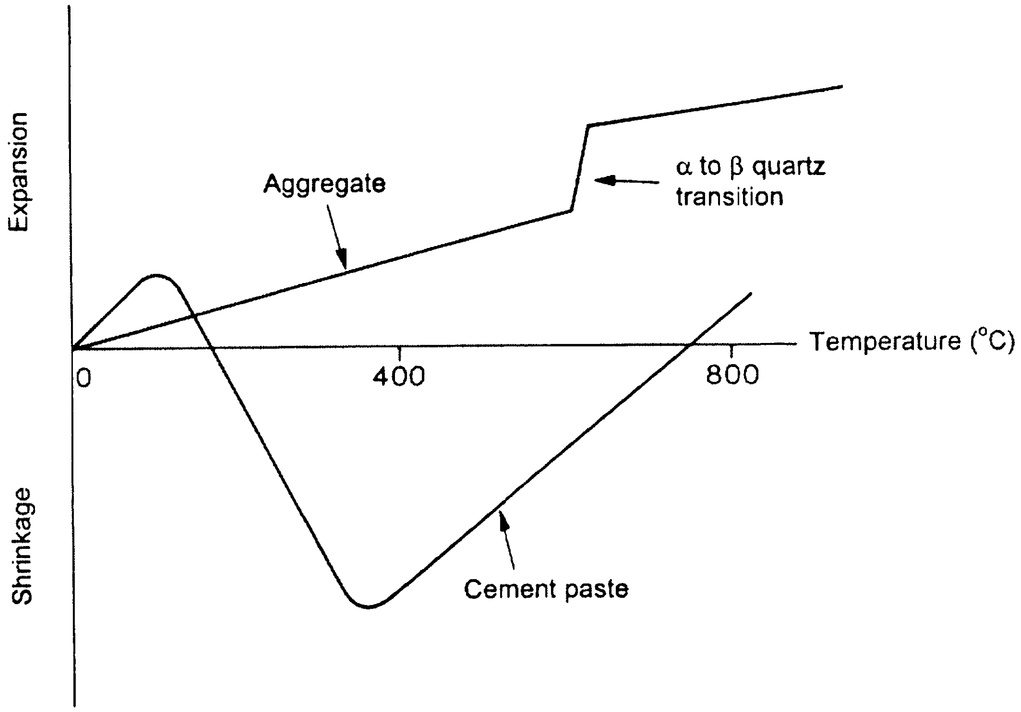


Figure 7.8: Thermal incompatibility between the aggregate and cement paste causing parasitic thermal stress the is to a large extent alleviated by LITS in compression.

7.4.3 Influence of temperature

Compressive strength reduces from ambient to reach a minimum at 80°C for unsealed concrete (Figure 7.9). This is an "apparent" strength loss, largely reversible upon cooling, and is attributed to the weakening of the physical van der Waal's forces as the expanding water molecules push the CSH layers further apart. This minimum, therefore, does not appear when the "*residual*" strength is measured after cooling and plotted against previous temperature. Frequently, the "*hot*" strength is shown to increase to a maximum at about 200-300°C greater than the initial strength prior to heating. Most concretes experience a strength reduction above 300°C, but this depends upon the type of aggregate and cement blend used in the mix (e.g. Figure 7.10). Between 300°C and 600°C there is room for improving the strength of concrete by judicial mix design. However, a marked increase in the "basic" creep of Portland cement paste and concrete occurs at about 550-600°C [KHO 86a] indicating this temperature to be critical, above which concrete is not structurally useful. Fortunately, in fire only the zone near the concrete surface experience temperatures greater than 300°C owing to the low thermal diffusivity of concrete in general. The normal practice after fire is to remove and replace this "overheated" layer. As long as the concrete does not spall during fire, this layer continues to provide thermal protection to the steel and inner concrete and would effectively act as a thermal barrier, although its structural role becomes greatly diminished.

Another important phenomenon, also not fully appreciated, is the major difference in behaviour between "*unsealed*" and moist "*sealed*" concrete at temperatures above about 100°C. The dominant process for unsealed concrete relates to the loss of the various forms of water (free, adsorbed and chemically bound), while the dominant process in sealed concrete relates to hydrothermal chemical reactions which could result in much weaker or stronger gel depending upon the CaO/SiO_2 ratio (i.e. C/S ratio). The C/S ratio is influenced by the use of cement replacements such as slag, pulverized fly ash (pfa), or silica fume in the mix.

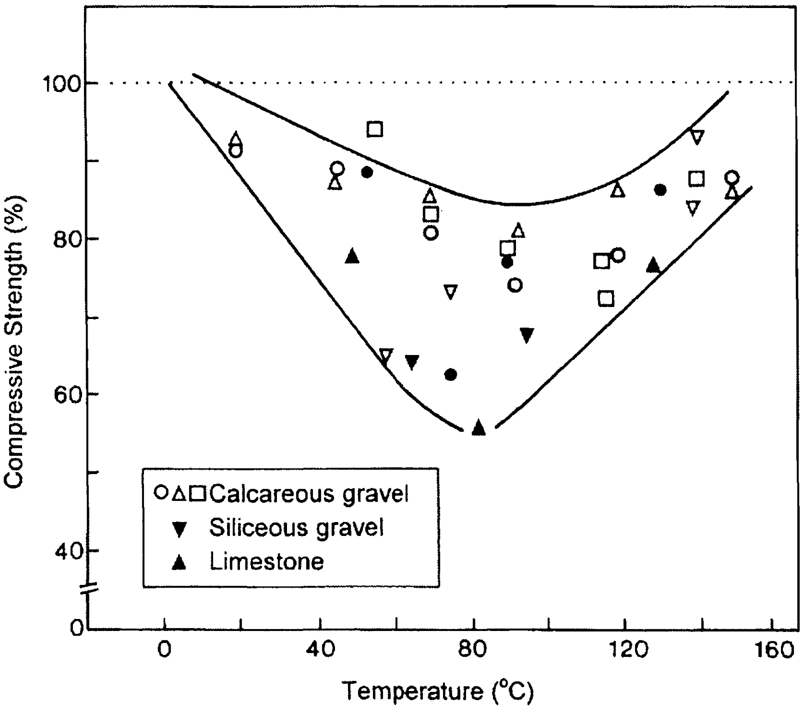


Figure 7.9: Common trend of unsealed concrete indicating a reduction in strength up to about 80°C owing the thermal expansion of the moisture - and a recovery in strength up to about 200°C owing to the moisture loss. Both influencing the physical cohesive forces in the C-S-H [BLU 76].

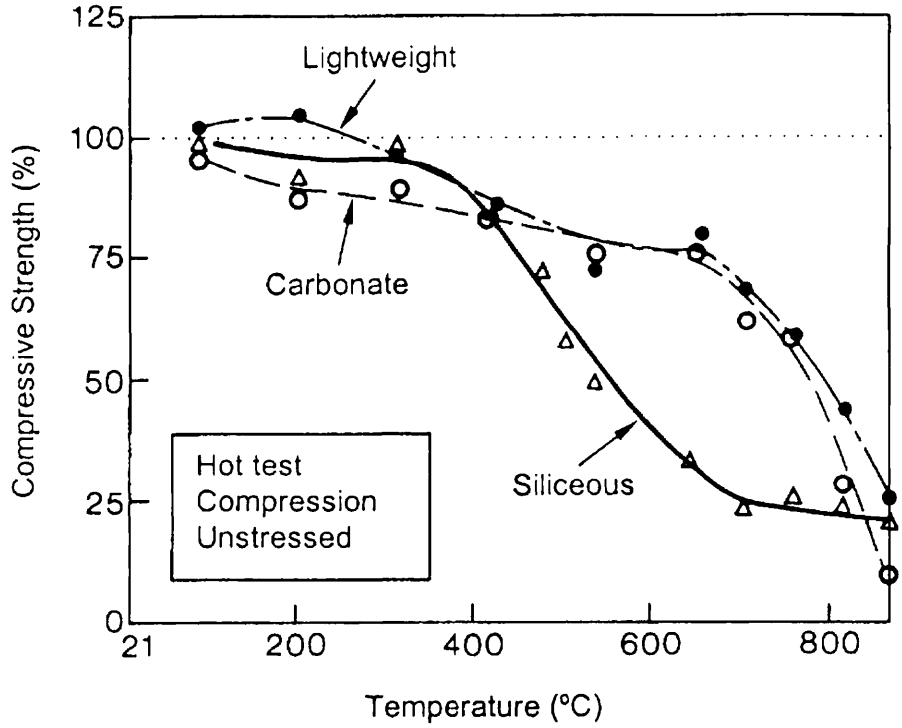


Figure 7.10: An example of the influence of aggregate type upon the compressive strength of unsealed concrete tested at temperatures but heated without stress [ABR 71].

Given the large number of material and environmental factors that influence the compressive strength of heated concrete, it is no surprise that measurements of compressive strength at 150°C can yield results ranging from as low as 30% to as high as 120% of the original cold strength (Figure 7.6). It is no wonder that data of concrete strengths at elevated temperatures taken from different sources differ substantially, and in several cases even seem contradictory. Therefore, representing "typical" strength behaviour of all "normal weight concrete" at high temperatures with a single "average curve" can be misleading unless the specific mix and environmental conditions (eg heating rate, load during heating, moisture condition, cooling rate etc.) are specified.

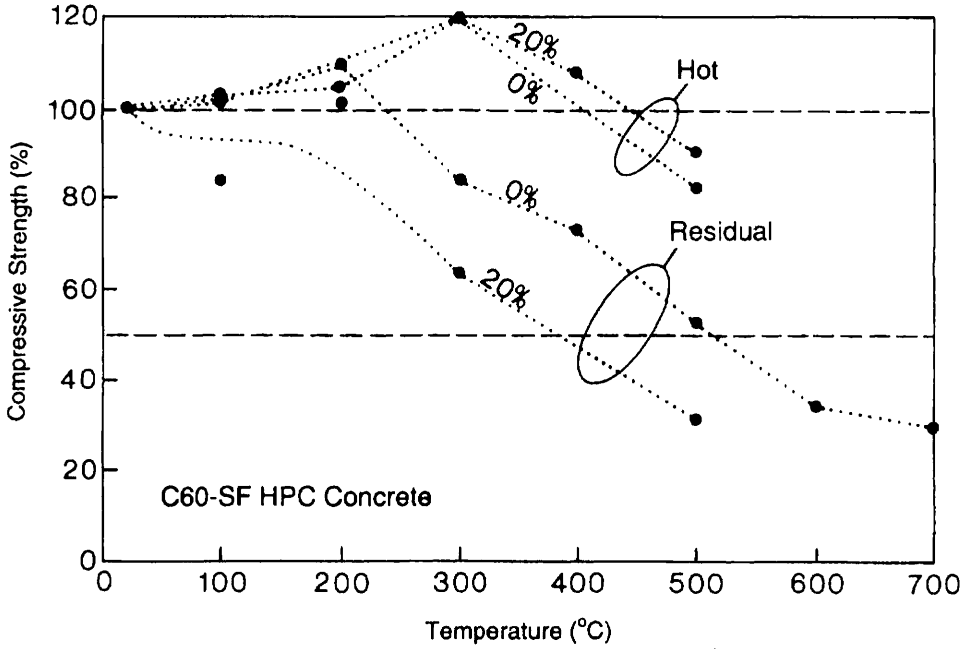


Figure 7.11: High strength concrete indicates higher hot than residual strength [KHO 99].

Recent research [KHO 99] has extended the testing of compressive strength from normal strength concrete to high strength concrete (HSC – strengths 60-100 MPa) and also ultra-high strength concrete (UHSC containing steel fibres – strengths 100-300 MPa). The results shown indicate that the residual strength declines more steeply than the hot strength of C60 HSC (Figure 7.11). The reverse is true for UHSC where the hot strength declines more steeply than the residual strength (Figure 7.12). The influence of steel fibres in the UHSC concrete may in part account for this interesting phenomenon

7.5 Mix design against strength loss and spalling

Deterioration of mechanical properties can be reduced for the temperature range from ambient up to 600°C by judicial design of the concrete mix (Figure 7.15-7.16). This would take into consideration the behaviours of (a) the aggregate, (b) the cement paste, and (c) the interaction between them. It also considers the processes that influence the strength of concrete during the four stages of heating, constant temperature, cooling and post-cooling (Figure 7.13). During first heating there are many processes that influence gain or loss of compressive strength (Figure 7.14).

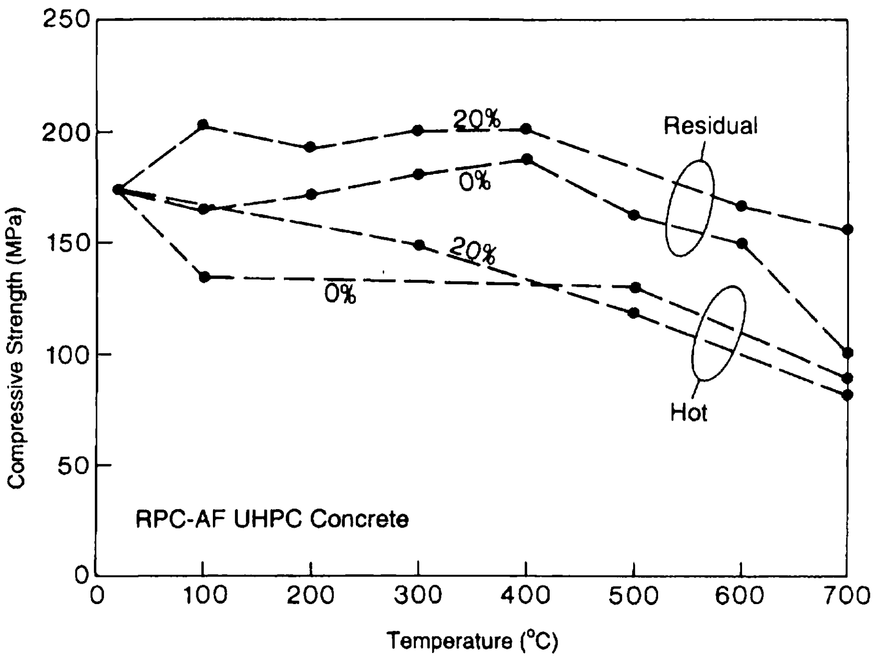


Figure 7.12: Ultra high strength concrete indicates higher residual than hot strength [KHO 99].

7.5.1 Aggregate

The choice of aggregate is probably the more important since some aggregates such as flint or Thames river gravel break up at relatively low temperatures (below 350°C) while other aggregates such as granite exhibit thermal stability up to 600°C. The thermal stability of different aggregates increases in the following order: Flint, Thames River gravel, limestone, siliceous, basalt, granite, gabro [KHO 97]. Other desirable features in aggregates are (a) low thermal expansion which improves thermal compatibility with the cement paste, (b) rough angular surface which improves the physical bond with the paste, and (c) the presence of reactive silica which improves the chemical bond with the paste.

7.5.2 Cement blend

As for the cement blend, an important feature is the C/S ratio. A low C/S ratio results in a low Calcium Hydroxide ($Ca(HO)_2$) content in the original mix and ensures a more beneficial hydrothermal reaction. Calcium Hydroxide is not desirable because it dissociates at about 400°C into CaO and CO_2. The CaO re-hydrates expansively and detrimentally upon cooling and exposure to moisture. The C/S ratio is reduced in practice by the use of slag, pfa or silica fume. Tests show that the use of slag produces the best results at high temperatures followed by pfa and then silica fume [KHO 90, SAR 93]. The relatively low performance of the silica fume cement paste (contrary to its high durability performance at room temperature) may be attributed to the dense low permeability structure of the paste which does not readily allow moisture to escape from the heated concrete thus resulting in high pore pressures and the development of microcracks.

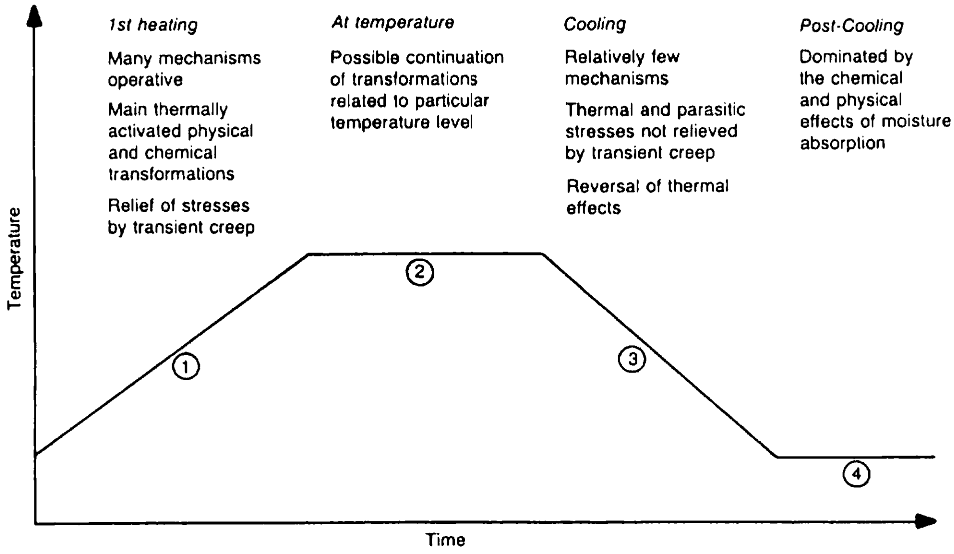


Figure 7.13: Factors influencing strength during the four stages of the heat cycle [KHO 92]

Mechanism	Temperature range °C	Gain (G) or loss (L) in strength
Aggregate thermal instability	Aggregate dependent	L
Thermal incompatibility and parasitic stresses (relaxed by transient creep)	20 – 600+	L
Changes in strength of bond shell	20	G or L^1
Aggregate-paste chemical reaction	20 – 200+	G or L^1
Increase in temperature of water-gel system	20 – 200 (^{4})	L
Accelerated hydration	20 – 90 (^{4})	G
Accelerated hydration	20 – 200 (^{4})	G or L (C/S ratio dependent)
Loss of free water	20 – 200 (^{4})	G
Loss of physically bound water increase in surface tension	80 – 200 (^{4})	G
Hydrothermal reactions	100 – 200 (^{4})	G or L (C/S ratio dependent)
Pore pressure stresses^5	100 – 200 (^{4})	L
Shrinkage stresses (relaxed by transient creep)	20 – 200 (^{4})	L
Changes in chemical binding on dehydration of C-S-H	100 -300+	G (?)
Increase in porosity	20 – 80 (?)	G (?)
Dissociation of $Ca(OH)_2$	390-450	L (perhaps insignificant)
Thermal effects	20-600+ (melting at 1350°C)	L^3
2nd stage breakdown of chemical bonds	600 - 750	L
Ceramic binding	800	Negligible hot strength but improved cold strength
Thermal stress cracking	20	L

^{1}Depends on mix constituents, ^{2}More pronounced above 300°C owing to cracking around $Ca(OH)_2$ and cement grains, ^{3}Significant above 600°C, ^{4}Mosture assumed lost at 200°C, ^{5}Although stresses relaxed by transient creep, explosive spalling can occur at high rates of heating

Figure 7.14: Factors causing loss or gain of strength of unsealed concrete during first time heating at approximate temperature ranges– relative importance not indicated [KHO 92].

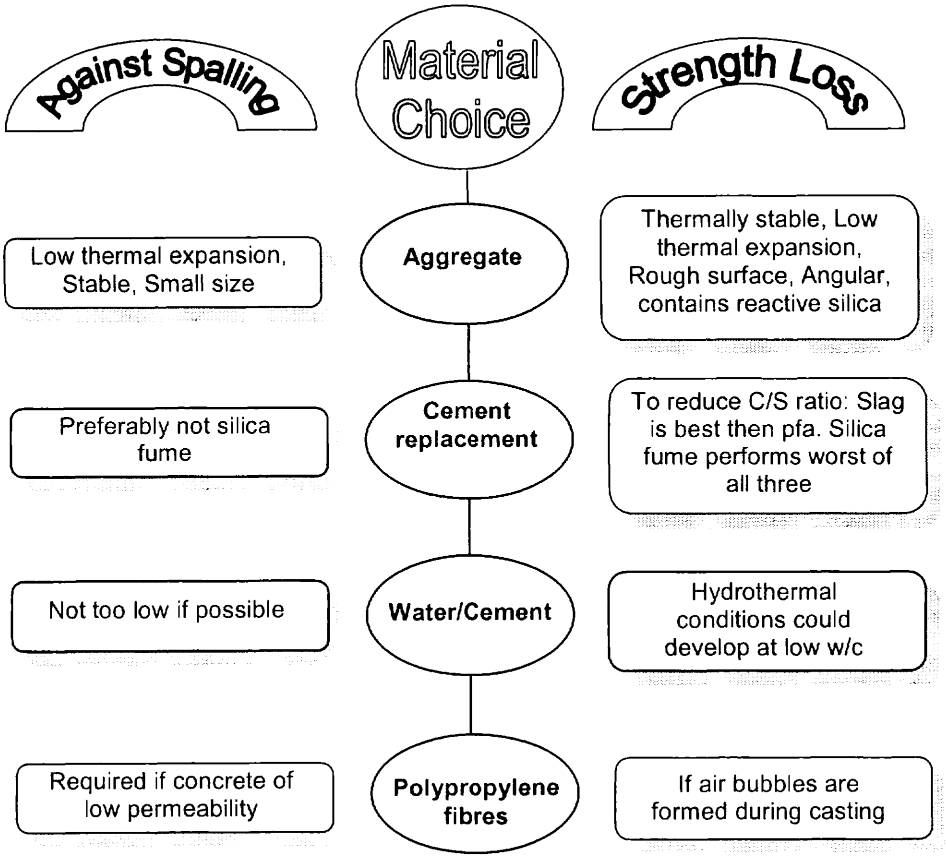


Figure 7.15: Concrete design again spalling and against strength loss [KHO 00].

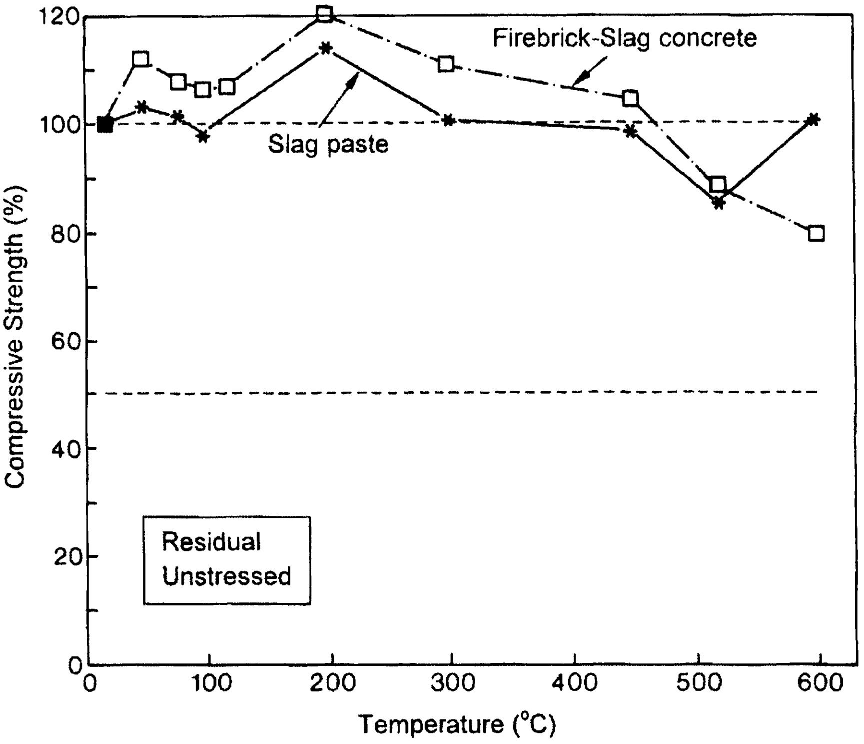


Figure 7.16: Concrete designed to demonstrate that a concrete can be designed without residual strength loss up to about 500-600°C. Two key factors here include thermally stable aggregate and the type of cement. Note that even with these characteristics, the basic creep of concrete (essentially of Portland cement paste) increases considerably in the "hot" state above about 500-600°C and thus is unable to bear load. [KHO 90, SAR 93]

This chapter has presented the key features of the mechanical behaviour of concrete at high temperature that provides assistance in the design of concrete for high temperature exposure and for input for numerical modelling. Modelling of concrete at high temperatures requires the input of rea listic materials properties data. Ignoring important materials factors in the model (e.g. effect of load upon the strain behaviour of concrete during first heating, the moisture condition, the types of aggregate and cement blend) could produce meaningless results.

8 References

[ABR 71] Abrams, M. *Compressive strength of concrete at temperatures to 1600F.* ACI Special Publication SP25, 1971, Detroit

[AKH 73] Akhtaruzzaman, A.A. *The effect of transient and steady state temperatures on concrete. PhD Thesis, Imperial* College, London, 1973.

[ALE 68] Alexander K. M. et. Al. *Aggregate-cement bond, cement paste strength and the strength of concrete.* Proc. Conf. structure of concrete and its behaviour under load. Cement and Concrete Association, London. 1968. pp 59-92

[ALO 99] M. Cruz Alonso & C. Andrade. *Microstructural Ana*lysis. CSIC Report to the HITECO project, 1999.

[ANC 86] Anchor, R.D., Malhotra, H.L. and Purkiss, J.A. (Editors). Design of structures against fire, *Elsevier Applied Science Publishers*, 1986, pp. 251.

[AND 73] Anderberg, Y. and Thelanderson, S., "*Stress and deformnation characteristics of concrete at high temperature: 1. General discussion and critical review of literature*". Lund Institute of Technology, Lund, Sweden, Bulletin 34, 1973.

[AND 76] Anderberg, Y. *Fire-exposed hyperstatic concrete structures. An experimental and theoretical study.* Lund Institute of Technology. Bulletin 55. Lund 1976.

[AND 92a] Anderberg, Y. and Pettersson, O. *Manual on Fire Engineering Design of Concrete Structures.* Swedish Board for Building Research, 1992. ISBN 91-540-5448-6, T13:1992, Part 1.

[AND 92b] Anderberg, Y. *Manual on Fire Engineering Design of Concrete Structures – Temperature Appendix.* Swedish Board for Building Research, 1992. ISBN 91-540-5448-6, T13:1992, Part 2.

[AND 98] Anderberg, Y. *Fire Engineering Design of Structures Based on Design Guides.* 2nd International Conference on Performance-Based Codes & Fire Safety Design Methods on Maui, USA, 7-9 May, 1998

[BAZ 76] Bazant Z. *Review of literature on high temperature behavior of concrete.* Report ORNL-TM-514s, Oak Ridge National Laboratory, Tenn, USA, pp 71-142, Jan 1976.

[BAZ 79] Bazant, Z. and Thonguthai, W. *Pore pressure in heated concrete walls - theoretical prediction.* Magazine of Concrete Research, 31(107), 67-75, 1979.

[BAZ 96] Bazant, Z. and Kaplan, M., *Concrete at high temperatures*, Longman Group Ltd., 1996, pp. 412, ISBN 0-582-08626-4.

[BEA 01] Breunese, A.J. *Tensile properties of concrete under fire.* TU Delft – TNO Centre for Fire Researc, Delft, The Netherlands, 2001

[BEI 02] Beitel, J., Iwankiw, N., *Analysis of Needs and Existing Capabilities for Full-Scale Fire Resistance Testing*, NIST report GCR 02-843, Gaithersburg, USA, 2002

[BER 72] Berterro, V. and Polikva, M. *Influence of thermal exposures on mechanical characteristics of concrete.* ACI Special Publication SP34, Detroit, 1972

[BER 73] Berwanger, C. and Sarkar, A. *Effect of temperature and age on thermal expansion and modulus of elasticity of concrete.* Behaviour of Concrete Under Temperature Extremes. ACI special publication SP39, 1973.

[BLU 76] Blundell, R., Dimond, C., Browne R. *The properties of concrete subjected to elevated temperatures.* CIRIA Underwater Engineering Group, June 1976. Proceedings

[BRO 68] Browne, R. *Properties of concrete in reactor vessels.* Proceedings of conference on Prestressed Concrete Pressure Vessels, paper c-13, pp. 131-151. The Institution of Civil Engineers, London, 1968.

[BS 85] British Standards Institution *BS8110 : Part 2 : 1985, Structural use of concrete. Code of practice for special circumstances. Section four, Fire resistance.* BSI, London, 1985.

[BUC 01] Buchanan, A.H., *Structural Design for Fire Safety*, University of Canterbury, New Zealand, 2001. John Wiley & Sons, Ltd.

[BUI 92] Building Regulations. *Approved Document B 3: Internal fire spread (structure)* H.M.S.O., The Building Regulations Approved Document B, Fire Safety, 1992 edition, pp. 131.

[CAM 66] Cambell-Allen and Desai D. *The influence of aggregate on the behaviouirs of concrete at elevated temperatures.* Nuclear Engineering and Design 2, 1966. 54-65, 246-255.

[CEM 79] Cembureau *Concrete for fire resistant construction* Cembureau Report, Published by the Cement and Concrete Association, 1979.

[CHE 01] Cheyrezy, M, Khoury, G. and Behloul, M. *Mechanical properties of four high-performance concretes in compression at high temperatures.* Revue Francaise de Ginie Civil, Volume 5, No. 8, 2001, pp. 1159-1180.

[CHR 72] Christiaanse, A., Langhorst, A. and Gerriste, A. *Discussion of fire resistance of lightweight concrete and spalling.* Dutch Society of Engineers (STUVO), Report 12, Holland, 1972.

[COL 75] Collet, Y. *Conductive thermique du materiau beton (thermal conductivity of concrete).* Group de travail "Beton leger de structure" et "Compartment du Materiau Beton on Fonction de la Temperature", April 1975.

[CON 95] Connolly, R.J. *The spalling of concrete in Fires* PhD thesis submitted to Aston University, April 1995, pp. 294.

[COP 79] Copier, W.J. *The spalling of normalweight and lightweight concrete on exposure to fire.*Heron, Vol. 24, No. 2, 1979, pp. 92.

[COP 80] Copier, W.J. *The spalling of normal weight and lightweight concrete exposed to fire.* American Concrete Institute publication SP-80. Paper SP 80-7, pp. 219-235.

[COX 02] Cox, G., *Fire Engineering-gas phase*, CIB Global Leaders Summit on Tall Buildings, BRE, Watford, 24 - 26 April, 2002.

[CRI 72] Crispino, E. *Studies in the technology of concrete under thermal conditions.* ACI Special publication SP34, Vol. II, Detroit, 1972.

[DIA 90] Dias, W.P.S., Khoury, G.A., and Sullivan, P.J.E., *Mechanical properties of hardened cement paste exposed to temperatures up to 700°C.* ACI Materials Journal, pp. 160-166, March-April 1999.

[EC1 91] Eurocode 1: *Basis of design and actions on structures, Part 2.2: Actions on structures exposed to fire, EN 1991-1-2:2002.* November 2002.

[EC2 05a] Eurocode 2: *Design of concrete structures, Part 1: General rules – Structural fire design EN 1992-1-2:2002.* February 2005.

[EC2 05b] Eurocode 2 - *Annex A. Design of concrete structures, Part 1: General rules – Structural fire design EN 1992-1-2:2002.* February 2005.

[FEL 04] Fellinger, J.H.H. *Shear and Anchorage Behaviour of Fire Exposed Hollow Core Slabs*, Ph.D. Thesis, TU Delft, Delft, the Netherlands, 2004.

[FIP 78] Federation Internationale de la Precontrainte *Recommendations for the design of reinforced and prestressed concrete structural members for fire resistance.* FIP/CEB Guides to Good Practice, Publication 15.336, 1978.

[GUO 87] Guo and X.-Q. Zhang. "*Investigation of complete stress-deformation curves for concrete in tension*". ACI Materials Journal, 84, No. 4, pp. 278-85 (1987)

[HAG 05] Hager, I.G. *Comportement a haute temperature des betons a haute performance – evolution des principales mecaniques* l'Ecole, 2005.

[HAR 65] Harmathy, T.Z. *Effect of moisture on the fire endurance of building elements.* ASTM special technical publication 385, Philadelphia, 1965.

[HAR 67] Harmarthy, T.Z. and Allen, L.W. *Thermal properties of selected masonry units concretes.* ACI Journal, no. 11, pp. 502-506, Nov. 1967

[HAR 70] Harmarthy, T.Z. *Thermal properties of concrete at elevated temperatures*, Journal of ASTM Materials, Vol. 5, No. 1, March 1970.

[HAR 73] Harmarthy, T.Z. and Allen, L.W. *Thermal properties of selected masonry units concretes.* ACI Journal, no. 70, pp. 132-42, 1973

[HER 84] Hertz, K.D. *Heat induced explosion of dense concretes.* Report no. 166, Institute of Building Design, Denmark, 1984.

[HRD 72] Harada, T., Takeda, J., Yamane, S. and Furamura, F. *Strength, elasticity and thermal properties of concrete* ACI Special Publication SP34, Detroit 1972.

[IDI 75] Idine, R., Lee, J. and Bresler, B. *Behaviour of reinforced concrete under variable elevated temperatures.* University of California, Fire Research Group UCB FRG 75-8, April 1975.

[ILL 73] Illston, J.M. and Saunders, P.D. *The effect of temperature change upon the creep of mortar under torsional loading.* Magazine of Concrete Research, 1973, 25, No. 84, 136-144

[ISE 75] Institution of Structural Engineers *Fire resistance of concrete structures.* Report of a Joint Conference of the Institution of Structural Engineers and the Concrete Society. ISE, London 1975.

[ISO 75] ISO 834, *Fire Resistance Test - Elements of building construction.* International Standard 834, 1975-11-01.

[JUM 89] Jumpannen, U-M. *Effect of strength on fire behaviour of concrete.* Nordic Concrete Research, Publication No. 8, 1989.

[KAL 97] Kalifa, P., Menneteau, D., and Sallee, H. Moisture and temperature distribution in a specimen heated on one face. CSTB, Annual report, 1997.

[KAP 72] Kaplan, M.F. and Roux F.J.P. *Effects of elevated temperature on the properties of concrete for containment shielding of nuclear reactors.* ACI special publication SP34, Detroit, 1972.

[KAP 89] Kaplan, M.F *Concrete radiation shielding*: Nuclear Physics, Concrete Properties, Design and Construction, Longman, London, 1989.

[KHO 83] Khoury, G.A. *Transient thermal creep of nuclear reactor concrete pressure vessel type concretes.* PhD Thesis, University of London, 1983.

[KHO 84] Khoury, G.A., Sullivan, P.J.E. and Grainger, B.N. *Radial temperature distributions within solid concrete cylinders under transient thermal states.* Magazine of Concrete Research, Vol. 36, No. 128, pp. 146-156, Sept. 1984.

[KHO 85a] Khoury, G.A., Sullivan, P.J.E. and Grainger, B.N. *Transient thermal strain of concrete: literature review, conditions within specimen and behaviour of individual constituents.* Magazine of Concrete Research, Vol. 37, No. 132, pp. 131-144, Sept. 1985.

[KHO 85b] Khoury, G.A., Sullivan, P.J.E. and Grainger, B.N. *Strain of concrete during first heating to 600°C under load.* Magazine of Concrete Research, Vol. 37, No. 133, pp. 195-215, Dec 1985.

[KHO 86a] Khoury, G.A., Sullivan, P.J.E. and Dias, W.P.S. *Deformation of concrete and cement paste loaded at constant temperatures from 140°C to 720°C.* Materials and Structures, Vol 19, No. 110, pp. 97-104, March 1986.

[KHO 86b] Khoury, G.A., Sullivan, P.J.E. and Grainger, B.N. *Strain of concrete during first cooling from 600°C under load.* Magazine of Concrete Research, Vol. 38, No. 134, pp. March 1986, pp 195-215.

[KHO 87] Khoury, G.A., Sullivan, P.J.E., Gungoosingh, R. and Malektojar, S. *Effect of age and specimen size on the transient thermal strain of concrete.* In Proceedings of the 9th International Conference on Structural Mechanics in Reactor Technology, Lausanne, Switzerland (Whitmann, F.H. (ed.)) Balkema, Rotterdam, 1987, Vol. H., pp. 133-139.

[KHO 90] Khoury, G.A., Sarshar, R., Sullivan, P.J.E., and Grainger, B.N. *Factors affecting the compressive strength of unsealed cement paste and concrete at elevated temperatures up to 600°C.* Proc. 2nd Int. Workshop on Mechanical Behaviour of Concrete under Extreme Thermal and Hygral Conditions, ISSN 0863-0720, pp. 89-92, Weimar, 1990.

[KHO 92] Khoury, G.A., *Compressive strength of concrete at high temperatures: a reassessment.* Magazine of Concrete Research, 44, No. 161, pp. 291-309, Dec 1992.

[KHO 97] Khoury, G.A., *Aggregate characteristics at high temperature,* HITECO Report , Imperial College, December 1997. pp 27

[KHO 99] Khoury, G.A., *Mechanical behaviour at high temperature in compression,* HITECO Report , Imperial College, August 1999. pp 72

[KHO 00] Khoury, G.A., *Effect of fire on concrete and concrete structures.* Progress in Structural Engineering and Materials, Volume 2, No. 4, Oct-Dec 2000, pp. 429-447.

[KHO 02a] Khoury, G.A., Majorana, C.E., Pesavento, F. and Schrefler, B.A. *Modelling of heated concrete.* Magazine of Concrete Research, 54, No. 2, April 2002, pp. 77-101.

[KHO 02b] Khoury, G.A. *Passive protection against fire. Tunnels & Tunnelling International,* November 2002, pp. 40-42.

[KHO 02c] Khoury, G.A. *Passive protection against fire* Tunnels & Tunnelling International, November 2002, pp. 40-42

[KHO 05] Khoury, G.A. *Effect of heat on concrete material,* Imperial College report, 1995, pp. 73.

[KHO 06a] Khoury, G.A. *Strain of concrete during two thermal cycles. Part 1: strain over two cycles, during first heating and at subsequent constant temperature.* Magazine of Concrete Research, 2006, 58, No. 6, August, 367-385.

[KHO 06b] Khoury, G.A. *Strain of concrete during two thermal cycles. Part 2: strain during first cooling and subsequent thermal cycle.* Magazine of Concrete Research, 2006, 58, No. 6, August, 387-400.

[KHO 06c] Khoury, G.A. *Strain of concrete during two thermal cycles. Part 3: isolation of strain components and strain model development.* Magazine of Concrete Research, 2006, 58, No. 6, September.

[KHO 06d] Khoury, G.A. NewCon Project. *Concrete Engineering International,* Spring 2006, pp 6-11

[KHO 06e] Khoury, G.A. Researching explosive spalling, Tunnels & Tunnelling International, June 06, pp41-45

[KHO 07] Khoury, G.A. *Spalling* Internal report, Imperial College, London, 2006. Chapter to be published in Book on "Effect of heat on concrete, 2008", Khoury, G.A. and Majorana, C.E.

[KIS 76] Kishitana, K. and Shiiba, H. *Thermal behaviour of cement paste in medium temperature zone.* Proc. 19th Japan Congress on Materials Research, Published by Soc. Of Natls, Sci. Japan, Kyoto, 1976.

[KOTT 79] Kottas, R., Seeberger, J. And Hisdorf, H. *Strength cahracteristics of concrete in the temperature range 20°C to 200°C.* Paper H1/2, 5th SMIRT Int. Conference, Berlin 1979.

[LAN 71] Lankard, D.R. et.al. *Effects of moisture on the structural properties of Portland cement exposed to temperatures up to 500°F.* ACI Special Publication SP25, Detroit, 1971.

[LEA 20] Lea, F., *The effect of temperature on some of the properties of materials, Engineering,* Engineering Vol. 110, 1920, pp 293-298. London 1920.

[LEA 22] Lea, F., and Stradling, R. *The resistance to fire of concrete and reinforced concrete.* Engineering, Vol 114, No 2959, London, September 1922.

[LEZ 67] Lezy, R. and Pailliere, A.A. *The improvement of concretes mortars and grouts by the addition of resin.* Rilem symp. on synthetic resins in building construction. Paris 1967.

[MAG 70] Magnusson, S.E. & Thelandersson. S. *Temperature-time curves for the complete process od fire development. A theoretical study of wood fuel fires in enclosed spaces.* ACTA Polytechnica Scandinavica, Ci65, Stockholm 1970.

[MAL 84] Malhotra, H.L. *Spalling of concrete in fires.* Construction Industry Research and Information Association, Technical Report No. 118, London 1984, pp. 36.

[MAR 72a] Marechal, J.C. *Thermal conductivity and thermal expansion coefficients of concrete as a function of temperature and humidity.* ACI Special publication SP34, Vol. II, Detroit, 1972.

[MAR 72b] Marechal, J.C. *Variations in the modulus of elasticity and Poisson's ratio with temperature.* ACI Special publication SP34, Vol. 1,Detroit, 1972.

[MAR 78] Martna, J. *Concrete for use in rock caverns for storing hot water.* 1st int. symp. On storage in excavated rock caverns. Pergamon. Oxford, 1978.

[MAS 80] Massaza, F. and Pezzuoli, M. *Cement paste-quartz bond in autoclaved concretes.* Peroc 7th intr congr on chemistry of cements, Sections VII-16, Edition Septima, Papris 1980.

[MEN 34] Menzel, C. A. *Strength and volume changes of steam cured Portland cement mortar and concrete.* Proc. Am. Conc. Inst. 1934, **31**, No.2, pp. 125-148..

[MEY 72] Meyer-Ottens, C. *The question of spalling of concrete structural elements of standard concrete under fire loading.* PhD Thesis, Technical University of Braunschweig, Germany, 1972.

[MEY 77] Meyer Ottens, C. *The behaviour of concrete structural elements in fires. Spalling of normal concrete elements under fire stress: causes and preventive measures.* Building Research Establishment, Translation 2058, 1977.

[MOR 88] Morris, W.A., Read, R.E.H. and Cooke, G.M.E. *Guidelines for the construction of fire-resisting structural elements.* Building Research Establishment Report BR 128, 1988, pp. 39.

[NAN 76] Nanstad, R. *A review of concrete properties for prestressed concrete pressure vessels..* Oak Ridge National Laboratory Publication ORNL/TM-5497, October 1976.

[NAS 66] Nasser, K.W. and Neville, A.M., *Creep of concrete at temperatures above normal.* Nuclear Engineering and Design. 4. pp. 90-90, 1966.

[NAS 79] Nasser, K. and Marzouk, H. *Properties of mass concrete containing fly ash at high temperatures.* ACI J., 1979, April, pp 537-550.

[NEP 69] Nepper-Christensen, O. and Nielsen T.P.H. *Model deterioration of the effect of bond between coarse aggregate and mortar on the compressive strength of concrete.* Proc. Am. Conr. Inst., 1969, **66**, No. 1, 69-72.

[NEV 95] Neville, A.M., *Properties of Concrete*, Longman Scientific & Technical, 1995.

[NIS 72] Nishizawa, N. and Okamura, H. *Strength and inelastic properties of concrete at elevated temperatures*, ACI Special publication SP34, Vol. 1,Detroit, 1972

[OHI 72] Ohigishi, S., Miyasaka, S. and Cida, J. *On properties of magnetite and Serpentine concrete at elevated temperatures fir nuclear reactor shielding.* Paper 57, ACI Special publication SP34, Vol. II, Detroit, 1972.

[PES 00] Pesavento, F., *Non-linear modelling of concrete as multiphase porous material in high temperature conditions*, Ph.D. thesis, University of Padova, Padova Italy, 2000.

[PHI 58] Philleo, R. *Some physical properties of concrete at high temperatures*, Proc. American Concrete Institute, Vol. 54, 1958

[PIA 84] Piasta, J. *Heat deformations of cement phases and microstructure of cement paste.* Materials and structures: research and testing. RILEM, Paris, 1984, Vol. 17, No. 102, pp 415-420.

[RAA 74] Raask, E. *The thermal and chemical stability of limestone aggregates.* Materiaux et constructions, Vol. 7, no. 42, 1974. (published in English).

[RIL 95] Rilem Committee TC 129 MHT, *Compressive Strength. Materials and Structures*, U. Schneider (Chairman and Lead Author), 1995, 28, 410-414.

[RIL 97] Rilem Committee TC 129 MHT, *Thermal Strain.* Schneider, U. (Chairman), Khoury, G.A. (Lead Author). Materials and Structures, March 1997, pp.17 -21.

[RIL 98] Rilem Committee TC 129 MHT, *Transient creep*. Schneider, U. (Chairman), Khoury, G.A. (Lead Author). Materials and Structures, Vol. 31, June 1998, pp 290-295.

[ROB 78] Robson, T.D., *Refractory concretes, past, present and future*. ACI publication SP 57-1, 1978, pp. 1-10.

[ROS 80] Rostasy et al. *Changes of pore structure of cement mortars due to temperature*. Cement and Concrete Research, 1980. 10, 157-164.

[RWS 98] RWS / Rijkswaterstaat (Ministry of Transport, Public Works and Water Management) (Tunnel Department) *Fire Protection for Tunnels* GT-98036 1998

[SAN 93] Sanjayanan, G. and Stocks, L. *Spalling of high strength silica fume concrete in fire*. American Concrete Institute Materials Journal, March/April, 170-173, 1993.

[SAR 93] Sarshar, R. and Khoury, G.A. *Material and environmental factors influencing the compressive strength of unsealed cement paste and concrete at high temperature.*. Magazine of Concrete Research, 45, No. 162, pp. 51-61, March 1993.

[SCH 81] Schneider, U. *Physical properties of concrete from 20°C up to melting*, Betonwerk + Fertigei – Technik, Heft 3/81

[SCH 82] Schneider, U., Diederichs, U. and Ehm, C. The effect of temperature on steel and concrete for PCRVs, Nuclear Engineering and Design, no. 67, pp245-258, 1982.

[SCH 02] Schrefler, B.A., Khoury, G.A., Gawin, D., Majorana, C.E. *Thermo-hydro-mechanical modelling of high performance concrete at high temperatures*. Engineering Computations, Vol. 19, No. 7, 2002, pp. 787-819.

[SER 77] Sertmehmetoglu, Y. *On a mechanism of spalling of concrete under fire conditions*. PhD thesis, King's College, London, 1977.

[SHO 65] Shorter, G.W. and Harmathy, T.Z. *Moisture Clog Spalling*. In Proceedings of Institution of Civil Engineers, 20, 75-90, 1965.

[SMI 78] Smith, P., Chapter 25 *Resistance to high temperatures*. ASTM publication STP169B. Significance of tests and properties of concrete and concrete making materials, 1978.

[THE 74] Thelanderson, S. *Mechanical behaviour of concrete under torsional loading at transient, high temperature conditions*. Lund Institute of Technology, Bulletin 46, Lund, Sweden, 1974.

[VER 72] Verbeck, G. and Copeland, L.E. *Some physical and chemical aspects of high pressure steam curing*. Menzel symp. On high pressure steam curing. ACI publication SP32, 1972. pp. 1-13.

[WAU 73] Waubke, N. V., *On one physical aspect of strength loss of Portland cement concretes at temperatures up to 1000°C* (in German) Heft 2, Techn. Univ. Braunschweig, Nov. 1973.

[YOU 82] Young, J. F. (Bazant ZP and Wittmann FH Ed.). *The microstructure of hardened Portland cement paste: Chapter 1 – creep and shrinkage in concrete structures*. Wiley, Chichester, 1982.

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