SP 80-4

Materials Dominated Aspects of Design for Structural Fire Resistance of Concrete Structures

By John W. Dougill

Synopsis The behaviour of concrete and reinforced concrete are examined in the context of rational design for fire resistance. Links with plasticity theory are noted together with the requirements for ductility in Structures designed using these concepts. It is concluded that current methods of rational design are most applicable when structural behaviour is determined by the action of the steel in tension at high temperatures. In considering the limitations of the method, doubts are expressed on extensions to compression loading and shear where the problems of detailing the reinforcement for adequate ductility are more difficult. The problem of local failure by spalling is also considered with different mechanisms being identified and results presented for pore pressures developed during heating. The paper concludes with a pragmatic philosophy for design in which a defensive view of bond, local damage and compression failure is to be adopted within an overall framework of rational design and knowledge from full scale tests.

Keywords: concrete slabs; continuous beams; fire resistance; pore water pressure; reinforced concrete; spalling; structural design; thermal expansion; thermal stresses.

John W. Dougill is Professor of Concrete Structures and Technology and Head of the Concrete Section in the Department of Civil Engineering, Imperial College, London. He serves as a corresponding member of ACI Committees 209 and 216 as well as RILEM and ASCE Committees.

INTRODUCTION

The aim of this paper is to look at structural fire resistance within the concept of rational design based on an ultimate load limit state. In particular, the aim is to identify ideal behaviour that would be required if these concepts were to be universally applicable and to compare this with that of concrete. To set the scene, it is useful to discuss the way in which concrete and steel reinforcement are affected by heating. It turns out that this behaviour is not ideal and that there must be reservations on the use of rational design techniques in respect of bond, shear and in the treatment of axial forces. This discussion leads to remarks on detailing and a general philosophy related to local damage and brittle failure.

THERMAL EXPANSION OF CEMENT PASTE AND CONCRETE

In dealing with thermal movements, it is useful to consider separately the properties of hardened cement paste and the aggregate.

There are two main effects that contribute to the thermal movement of a cement paste (1,2). These are the thermal expansion of the solid phase and the shrinkage that accompanies desorption of moisture during heating. For a moderate rise in temperature, the shrinkage is small and paste shows a net expansion. At higher temperatures, the effect of shrinkage outweighs the effects of thermal expansion so that there is a net contraction. Shrinkage continues with increasing temperature until, when only the most strongly bound water molecules remain, the paste begins to expand once more. The behaviour is not reversible, as resorption cannot occur during cooling unless free water is made available. Thus, the thermal movement of cement paste for moderate rates of heating is typically in the form shown in Figure 1, from Crowley (3).

It is possible to consider the thermal movement of cement paste to be the sum of reversible and irreversible components which are respectively due to thermal expansion and moisture desorption(4). However, it should be noted that no single relationship connecting strain with temperature exists for cement paste and that the length of the drainage path and the heating rate will also be influential. Thus, for fast rates of heating the expansion component becomes more important than at the slower rates when moisture loss and shrinkage may be very significant.

The aggregates used in concrete seldom exhibit shrinkage. Nevertheless, the range of behaviour during thermal expansion is quite varied and often complicated by phase changes (in quartzitic materials for instance) or decomposition of the aggregate, as occurs with limestone at very high temperatures.

With the dominant effect of shrinkage in the cement paste, the thermal movement of a concrete tends to be intermediate between that of the paste and the aggregate as shown in figure 2 from Harada (5). Due to the difference in thermal movement of the constituent materials and restraint to the expansion of the aggregate, stresses are developed in the cement paste and aggregate (1,4). If the temperature is less than around 300°C, these stresses are not enough to cause significant cracking and the concrete remains intact. There is then little loss of stiffness on heating and after cooling there is residual contraction effectively due to drying shrinkage in the cement paste.

At higher temperatures, the stresses developed due to thermal incompatibility are greater and can cause cracking. Breakdown occurs mainly between the aggregate and the cement paste. Because of this, the cement paste is not so effective in restraining the expansion of the aggregate as when cracking is absent. Thus, at high temperatures, cracking and loss of continuity in the concrete structure provide additional expansion and cause the overall thermal movement to approach nearer to that of the aggregate. The strains due to cracking are effectively irreversible and so counteract the drying shrinkage. Thus, cooling after high temperature exposure can lead to a large residual expansion as shown in figure 3.

The description given of the mechanisms affecting the gross thermal movement of concrete is somewhat simplified. For instance, no account has been taken of the effects of vapour pressure or pore water pressure on the induced strains (6). Nevertheless, it is clear that, even in the absence of load or restraint, the thermal movement of concrete is a complex phenomenon which will be difficult to describe in theoretical terms. However, such a description would not necessarily be useful as the strains due to heating and loading are inextricably linked due to the influence of cracking on the total deformation.

BEHAVIOUR UNDER LOAD AND RESTRAINT

In the more conventional laboratory tests, concrete is maintained at high temperature for a period and then loaded to failure. The main result is that the form of the complete stress-strain curve in compression is as observed at normal temperatures although the peak stress (i.e. strength) is reduced by heating and the strain corresponding to the peak stress increases dramatically at high temperatures. Furumura's (7,8) results show this particularly well and suggest that although brittle behaviour in compression can be expected at moderate temperatures, concrete becomes mechanically more stable at high temperatures.

Even this result has to be treated with caution. At normal temperatures, the form of the stress strain curve is largely determined by the development of micro-cracking under increasing strain. Similarly, at high temperatures, it seems that loss in strength and stiffness caused by heating is primarily due to cracking at a dimensional level less that representative of the material. Consequently, we would expect the effects of heating to be much reduced if concrete is heated under load or restraint of a sort that inhibits the development of micro-cracks.

For concrete with aggregate stiffer than the cement paste, a uniaxial compressive stress induces average stresses in the matrix which are entirely compressive (9). These would inhibit cracking so it is understandable that, when a normal weight concrete is heated whilst under a compressive stress, the reduction in strength and stiffness is much less than if the load were absent. Abrams (10) results shown in figure 4 follow this pattern. With only a small load, the strength of concrete with a carbonate aggregate is maintained at its normal value up to 700°C. The siliceous aggregate concrete does not behave so well, presumably due to the effects of the α–β quartz phase change which is accompanied by expansion. Nevertheless, the effects of the superimposed load are still significant. With the lightweight aggregate, the effect of superimposed load is different as it would tend to cause a tensile component of stress in the paste which would increase the tendency to micro-cracking.

The other property that should be noted, in the context of behaviour under load at high temperatures, is creep. This is very much accelerated by heating. Some idea of this follows when it is realised that in Cruz's tests (11) the creep achieved in five hours at 480°C was about the same as might be expected after a year at normal temperature. Creep has an important effect on structural behaviour both by rapid relaxation of stress caused by heating under restraint and by increasing the deformation under an applied load. These effects are most apparent when we consider conditions in which the temperature and loading change with time.

When concrete is heated under load, the thermal movement is heavily influenced by creep. This is evident from Fischer's (12) results shown in Figure 5. The contrast between the large residual contraction, afforded by a loaded concrete specimen heated to high temperature and then cooled, and the situation shown in Figure 3 is particularly noteworthy.

Another common situation arises when the free thermal movement is prevented or partly prevented by restraint afforded by a neighbouring structure or component. A typical example is provided by a wall panel or a floor in which the in-plane deformations are restrained by a surrounding building frame. In this situation, a rise in temperature induces compression in the panel. The rate of heating is important as far as the influence of creep is concerned. With fast heating, there is little opportunity for either relaxation or drying shrinkage so that, with sufficient restraint, the induced forces can be substantial. With slower rates of heating, relaxation becomes more important, the induced load is less and may even peak whilst the temperature continues to rise (13).

MODELS FOR CONCRETE BEHAVIOUR

This discussion of concrete behaviour has been developed in qualitative terms in order to emphasise the mechanisms involved and to identify the phenomena that would need to be included in a complete description of material behaviour. Already a number of workers have attempted limited descriptions for concrete in compression subject to high temperature exposure (14,15,16). Usually the approach is by an extension of procedures used at normal or only slightly elevated temperatures when cracking is insignificant and behaviour may be assumed to be linear. In this way, for instance, Anderberg et al (16) consider the total strain to be the sum of an instantaneous strain due to stress, the free thermal movement, creep and a correction term included to take account of the additional strain that occurs during temperature change. Agreement with experimental results is good for the rates of heating used with small laboratory samples. In spite of this, there must be some uncertainty with models of this sort when used with the fast rates of heating that occur in concrete sections exposed to fire. Equally important, the forms of model currently being adopted appear to ignore degradation or micro-cracking as a major influence on the stress-strain law. This seems to be a fundamental flaw: micro-cracking is known to be important at normal temperatures and is taken into account in the most recent general theories of material behaviour for use at normal temperatures (17,18). It is tempting to suppose that it is these theories that should be extended to take account of temperature effects.

These remarks are not intended to be critical of the development of analytical and computer methods for examining the effects of fire on concrete structures. The point being made is that such approaches are still at an early stage of development. They have their use in demonstrating phenomena (19) and exploring modes of structural behaviour (15,22). On the other hand, it is doubtful whether they are yet sufficiently reliable to provide quantitative data in situations when calibration with results from full scale furnace tests is not possible.

REINFORCEMENT

A considerable amount of information has now been accumulated on the properties and behaviour of reinforcement at high temperatures (16,23). This depends on the type of steel and the method of manufacture. Exposure to temperatures below 250°C has little effect on the strength or stiffness of reinforcement other than there is sometimes a small increase in yield stress due to stress relief. Higher temperature exposure causes a reduction in Young's modulus, yield stress and ultimate strength. For reinforcing bars of various types the nominal yield stress is reduced to around 50% of its normal value at temperatures in the range 550-600°C. With prestressing wire effects are more severe with a 50% loss in strength occurring at temperatures nearer 450°C.

Quite importantly, the sharp well-defined yield point for mild hot rolled steels disappears at high temperatures with strain hardening occurring after first yield to strains of 2% or more. Continuing strain hardening is not such a feature of the behaviour of cold drawn prestressing steel although this also shows considerable elongation before failure at high temperature. Again creep is appreciable and must be taken into account when a full analysis is attempted involving compatibility of deformations. Various procedures are available for estimating the creep deformation (16).

DESIGN

Currently, design for fire resistance implies limiting heat transmission and maintaining structural integrity or stability for a structure subjected to a standard exposure condition extending over some specified period. The period of exposure is decided from the nature of the contents and the use of the structure. The standard form of exposure, originally derived from burn out tests on structures (24), provides the link with the fire tests normally used for structural components.

Traditionally, design for fire resistance has been by interpolation between results from standard fire tests on single members carrying their maximum permitted working load. In these terms, design involves choosing suitable overall dimensions for members and providing adequate cover and supplementary reinforcement. Recommended values of cover and thickness are tabulated in Codes of Practice so the process is straightforward and convenient in imposing few demands on the designer.

The fire test is central to this approach. However, the usual form of fire test limits the amount of information and choice available to the designer. Investigations of the effect of continuity and restraint undertaken at the Portland Cement Association Laboratories and continued at Braunschweig highlighted these deficiencies by showing major differences in behaviour between continuous members and the separate elements normally tested. These results led directly to attempts to analyse the full behaviour of structures under fire conditions and to alternative methods of design based on behaviour likely to occur in a structure as opposed to that in furnace tests. This approach stems directly from the work at P.C.A. by Carlson, Selvaggio, Gustaferro, Abrams and others and has been further developed by Gustaferro (25) under the banner of "Rational Design". This approach has greatly influenced a number of recent reports (26-28) dealing with design for fire resistance.

RATIONAL DESIGN

The basis of rational design is seen most clearly in the treatment of flexural members. Consider the fire resistance of a simply supported slab spanning in one direction. The slab carries a uniformly distributed load W over a span L and before heating has a load factor γ . At this stage, the moment of resistance at the centre of the slab is

$$\frac{\gamma W L}{8} = A_s f_y \ell_a$$

where A_s is the area of steel, ℓ_a is the lever arm and f_y is the yield stress of the steel.

If the slab is heated from below, the temperature of the steel rises reducing the yield stress to $\Psi_s f_y$ where the coefficient Ψ_s is a function of the steel temperature.

The compression zone is in the upper cooler part of the slab, is kept under stress, and so suffers little or no deterioration in strength. Because of the reduction in yield stress, the lever arm at failure increases slightly; but we shall ignore this in noting that the slab survives if

$$\frac{WL}{8} \leqslant A_s \Psi_s f_y \ell_a \tag{2}$$

or, on combining this with (1),

$$\Psi_s \gamma \geqslant 1 \tag{3}$$

Equation (3) demonstrates the trade off, made possible by rational design, between strength and fire resistance. If more steel is used, the original load factor is increased and Ψ_s must be smaller at collapse indicating a longer period of exposure. This trade off is not apparent in using the results in fire tests in which the load factor γ is closely bounded for different forms of construction. Because of this, failure of fire tests tends to occur at a particular value of Ψ_s leading to the idea of a critical steel temperature. It is one of the principal results of rational design that this over restrictive concept has been discredited (25).

To turn the analysis into a design process, all that is needed are graphs,

(a) giving Ψ_s as a function of temperature and

(b) giving the temperature achieved in slabs heated in the standard manner. Such information is now generally available for a wide range of section shapes and reinforcement types (28).

The procedure is obviously capable of extension to continuous structures. Consider for example the multi-span beam shown in figure 6. An interior span, length L, carries a distributed load W and is subject to a fire from below. Before heating, the absolute value of the moments of resistance at the centre of the span and at the supports at M_c^O and M_s^O respectively. Thus, if the load factor existing before the occurrence of the fire is γ,

$$M_c^O + M_s^O = \gamma WL/8 \tag{4}$$

The effect of the fire is to change the moments of resistance to M_c^T and M_s^T (the superior T indicating a value at elevated temperature). Thus, failure is avoided as long as

$$M_c^T + M_s^T \geqslant WL/8 \tag{5}$$

Behaviour at the centre of the span will be similar to that of a simply supported slab. Thus, it is a good approximation to write

$$M_c^T = \Psi_s M_c^O \qquad (6)$$

Now, on defining a restraint, or detailing, factor

$$r = M_s^O / M_c^O$$

the condition for the beam to survive becomes

$$\Psi_s \gamma \geqslant 1 - r\,(\gamma M_s^T / M_s^O - 1). \qquad (7)$$

When r = 0, the result (3) for the simply supported beam is obtained. If $M_s^T/M_s^O = \Psi_s$, the proportional loss in moment capacity is the same at the supports as at midspan, so that the same criterion for survival (3) is obtained. However, the tensile reinforcement at the supports is well away from the effects of heating and, although the concrete in compression is subjected to the direct effects of fire, the concrete is heated under load and so likely to sustain the minimum loss in strength. Because of this, the proportional loss in moment capacity at the supports is likely to be much less than at mid-span. It follows that the fire endurance of a continuous beam will be greater than that of a single span beam having the same original load factor.

The inherently good performance of continuous beams is even more plainly evident when it is noted that failure cannot occur in flexure if the right hand side of (7) is negative. That is, the beam will apparently survive indefinitely if

$$\frac{M_s^T}{M_s^O} \geqslant \frac{1+r}{\gamma r} \qquad (8)$$

This condition can be satisfied by everyday designs that make no special allowance for fire resistance. For instance, with r = 1.5 and γ = 2.5, a one third reduction in moment capacity can be tolerated before the condition is violated. To put this in perspective, Carlson and Gustaferro (29) found only a 10% reduction in resistance moment at the support of a 305mm by 458mm deep rectangular reinforced concrete balanced cantilever beam after more than three hours exposure. Clearly continuous beams and slabs have intrinsically high fire resistance. This capability is lost only when poor understanding of structural behaviour under fire conditions leads to inadequate detailing of the reinforcing steel.

The continuous beam example focusses attention on moment capacity at the supports and the behaviour of concrete in compression. The view here is that, provided local damage by spalling does not occur, any loss in moment capacity will be small. Published recommendations (27,28) on rational design take a more conservative view. These suggest that the capacity of the compression zone should be calculated knowing the temperature at each level and associating with this the peak stress or strength of concrete obtained from laboratory tests. This procedure gives an impression of accuracy which is not justified. The strength values used are those from tests on concrete heated without any load present. More important, simultaneous use of peak values from the stress-strain curves for different temperatures implies a particular pattern of deformation within the beam which will not, in general, occur. It would be a fortunate but unlikely coincidence if the errors involved in these two operations cancelled. Neither assumption is soundly based and the effort involved in detailed analysis of the compression zone has only cosmetic value in justifying a particular choice of section.

In passing, if a reasonably complete constitutive law for concrete was available, it would be a useful exercise to analyse behaviour in the compression zone in a heated beam and deduce equivalent lever arm and stress-block factors for use in design. As yet this is not possible. Current models of material behaviour do not include an account of the effects of micro-cracking and cannot describe the beneficial effects of load during heating observed by Abrams(10) and Malhotra (30). Eventually though, the full analytical method should make a major contribution in investigations of structural details of this sort.

RATIONAL DESIGN AND PLASTICITY THEORY

Rational design, as used in analysing behaviour of beams, is a conventional extension of ultimate load theory developed for reinforced concrete. Formally, its justification rests on the correspondence between the materials and procedures used and the requirements of the Upper Bound Theorem of Plasticity. This applies for materials and structures exhibiting perfectly plastic behaviour. For these, the plateaux in the stress/strain or moment/curvature relations enables the statics and kinematics of a problem to be uncoupled with consequential concentration on one or other aspect.

In the kinematic approach, an assumption is made of the mode of collapse. An estimate of the collapse load follows when the energy dissipated in plastic deformation is equated to the work done during a virtual displacement corresponding to the assumed collapse mechanism. The Upper Bound Theorem shows that this estimate is either too high or correct, depending on the choice of collapse mode. The Theorem