EFFECTS OF ELEVATED TEMPERATURE
ON BOND IN
REINFORCED CONCRETE

by

F.D. Morley BSc

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This thesis is the result of research work for the degree of doctor of philosophy undertaken in the Department of Civil Engineering and Building Science, University of Edinburgh.

It is declared that the thesis has been composed by the author himself, and all work and results in the thesis have been carried out and achieved solely by him under the supervision of Dr. R. Royles, unless otherwise stated.

Edinburgh, October 1982

Paul D. Morley

P.D. Morley
ABSTRACT

The practical situation which led to this investigation is the case of fires in reinforced concrete structures. Concrete is a good fire resistant material and after a fire has finished the structure itself can often remain standing. Therefore a choice is available between reinstating the fire damaged structure as opposed to demolishing and reconstructing the whole of it. Often the former alternative can be quicker and cheaper, however, before reinstatement can begin it must be established that the damaged structure is suitable for such treatment. Hence an assessment of its residual capacity for structural performance must be made. To do this knowledge of the properties of steel and concrete and the bond between them at temperatures experienced in fires must be known.

Such information on the two materials themselves is readily available, however, little is known about the response to heat treatment of the bond properties between them. Therefore the aim of this project was to investigate the effect of heat on the bond properties between the steel reinforcing bar and the concrete.

To carry out this aim a review was made of heat effects on steel and concrete, within the context of fire resistance and reinstatement design. Also existing work on bond at ambient and elevated temperature was studied enabling the form of experimental procedure to be established.

The experiments themselves were on simple pull-out type specimens and looked at various parameters affecting the bond. From the results the effect of the bond reduction with respect to the anchorage and cracking in beams was examined. This enabled the bond reduction, on subjection to heat treatment, to be quantified for possible application to the reinstatement and fire resistant design processes.
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A  
A_0  
A_1  
A_2  
A_3  
B  
c_1  
c_2  
k  
k_1  
k_2  
m_1  
m_2

constants

a  
A_c  
A_s  
AER  
A_{nr}  
A_r  
c  
CSR  
d  
d_1  
D

rib height  
area of concrete  
area of steel  
acoustic emission ratio - emission as a proportion of the total accumulated emission counts at maximum bond stress  
area not resisting shear w.r.t. concrete tooth between ribs  
area resisting shear  
rib spacing  
concrete compressive strength as a proportion of the cube crushing strength at ambient temperature  
bar diameter  
diameter of the central hole in a bond test specimen  
overall diameter of a bond test specimen
\( E_c \) modulus of elasticity of concrete
\( E_s \) modulus of elasticity of steel
\( l \) length or height of cylindrical bond test specimen
\( l_v \) embedment or bond length
\( m \) modular ratio
\( M_u \) ultimate bending moment capacity
\( n \) number of ribs
\( P \) force
\( t \) time
\( T \) temperature
\( T_i \) concrete/steel interface temperature
\( T_f \) furnace temperature
\( T_m \) mean bond test specimen temperature
\( u \) mean bar perimeter
\( V \) shear force
\( x \) distance
\( x_1 \) distance from zero datum to first crack
\( z \) lever arm
\( \Delta \) displacement or slip
\( \Delta_o \) initial slip; slip at distance \( x = 0 \)
\( \Delta_x \) slip at a distance \( x \) from zero datum
\( \varepsilon_c \) concrete strain
\( \varepsilon_s \) steel strain
\( \mu \) ratio of concrete area to steel area
\( \sigma_b \) bond stress
\( \sigma_{bp} \) bond stress of plain bar (sliding resistance)
\( \sigma_{bx} \) bond stress at a distance \( x \) from zero datum
\( \sigma_c \) concrete stress

(ix)
$\sigma_{cn}$ concrete compressive strength beneath a rib and normal to the cross section of a bar

$\sigma_{cs}$ concrete shear stress in a tooth between ribs

$\sigma_{ct}$ maximum tensile strength of concrete

$\sigma_{cu}$ concrete compressive cube strength

$\sigma_{cx}$ concrete stress at a distance $x$ from zero datum

$\sigma_s$ steel stress

$\sigma_{sx}$ steel stress at a distance $x$ from zero datum

$\sigma_{sx_0}$ initial steel stress; steel stress at distance $x = 0$

$\sigma_{sx_1}$ steel stress at first crack at a distance $x_1$ from zero datum
CHAPTER 1

FIRE DAMAGE OF REINFORCED CONCRETE STRUCTURES
AND IT'S ASSESSMENT

1.1 GENERAL

1.1.1 Introduction

The nature of concrete is such that it is inherently fire resistant. When care is taken in the design of a concrete structure, even after a severe fire, the building often will not only remain standing but also will be able to be reinstated. Under favourable conditions the fact that the building is still standing and, due to reinstatement methods, can be back in use relatively quickly(1) keeps the losses involved to a minimum. Hence the reinstatement of concrete structures is often a viable alternative to demolition and reconstruction.

1.1.2 The Effect of Fire

The effects of fires in buildings can be examined under three headings (2),

(a) the combustibility and flammability of the material and its contents,
(b) the hazards to human life; and,
(c) effects on structural performance.

These points can be considered with reference to reinforced concrete in the following way.

(a) Reinforced concrete, being a fire resistant material, does not support combustion, rather it prevents combustion spreading to other parts of the building and to adjoining buildings. The reason for this is twofold,
Firstly concrete retains its structural integrity and so seldom collapses in a fire. The second point is that the thermal properties of concrete are such that the unexposed surface remains relatively cool. This prevents any material on the other side heating up and reaching flash point thus restricting the spread of the fire. Examples of fires in apartment buildings have been reported in which the fire was prevented from spreading from the apartment of origin to those directly below and above by concrete floors (3).

(b) Surveys on fires in buildings have shown death is, in the majority of cases, caused by suffocation from smoke and fumes (2). This implies that the type of construction does not greatly influence the safety risk assuming the structure has the minimum standards of fire resistance specified in the building regulations. However it has already been stated that even the most severe fires rarely cause total collapse of concrete structures. This helps to minimise the risk of injury due to structural collapse, especially for firemen who enter the structure and attack the fire from the inside (1).

(c) It has been seen that reinforced concrete has beneficial effects on (a) and (b), above. However it is apparent that this results from the good structural performance of concrete exposed to a fire. Therefore during a fire reinforced concrete is mainly involved in the structural performance of the building.

1.2 THE EFFECT OF HEAT ON THE STRUCTURAL PERFORMANCE OF REINFORCED CONCRETE.

Whilst considering the effect of fire on the structural performance of the material a threefold approach is pursued.

(a) The effect on the constituent materials; in this case concrete and steel,
the effect on individual members including the bond between the steel and concrete, and

(c) the effect on the overall structure.

1.2.1 The Effect of Heat on the Constituent Materials

1.2.1.1 Concrete

Concrete itself consists of cement, aggregate and water. In considering the effect of heat on its properties the different way that cement and aggregate react is very important.

1.2.1.1.1 Cement: - To obtain a suitable workability more water is added than is necessary for the hydration process. After the cement paste has hardened the water exists in three different states (4). There is free water in the space outside the range of surface forces, water physically absorbed on the gel surfaces, and chemically combined water forming part of the hydrated compounds. When cement paste is subjected to heat, the water is gradually lost. It is not removed at one particular temperature. The different compounds within the cement paste lose their water of crystallisation at different temperatures. Also the energy required to release the physically absorbed water varies so that water is being continually expelled from the paste throughout the whole temperature range. There are basically three operations which result in the removal of water from the hardened cement paste.

(a) Free water starts to evaporate at approximately 100°C.

(b) The tobermorite gel (1.62CaO·SiO₂·1.5H₂O) begins dehydrating at around 100°C, reaches its peak at 200°C and then decreases. This is followed by an increase in dehydration at 700°C.
Between 400°C and 600°C rapid dehydration of the CaO·H₂O component (calcium hydrate) takes place.

The effect of this loss of water is to cause shrinkage within the cement paste. This is illustrated in Fig. 1(5).

1.2.1.1.2 Aggregate:— The type of aggregate used strongly affects the performance of concrete when subjected to elevated temperatures. This is due to the variation in thermal conductivity, thermal diffusivity and the coefficient of thermal expansion of differing aggregates. The thermal conductivity and thermal diffusivity affect the rate of heat transmission from the hot surface to the unexposed surface, while the thermal expansion affects the cracking and spalling characteristics.

It is the effect of the thermal expansion on the cracking of the heated concrete that is of particular relevance here. Siliceous aggregates expand by the greatest amount. At around 575°C in particular rapid expansion occurs caused by the transformation of α-quartz to β-quartz. Calcareous aggregates (e.g. limestone) have more favourable thermal properties than siliceous aggregates, not only is their thermal expansion smaller, their conductivity is also less giving them an advantage in the fire situation. Therefore different aggregates behave with varying degrees of expansion and heat transmission properties making some more suitable for fire resistance purposes than others. However it is noted that the general effect of heat on most aggregates is to cause an expansion to take place.

1.2.1.1.3 Cracking:— It has been seen that when concrete is heated the cement paste shrinks while the aggregate expands. These different volume changes related to temperature variations cause cracking within the concrete to occur.
Concrete essentially consists of aggregate held together by a network of mortar. The mortar consists of fine grains of sand stuck together by the Portland cement. So the concrete can be considered to consist of a network of cement holding together stones of various size. If, in the concrete, all the stones are covered in cement no increase in total volume can take place unless the cement itself expands. Also, unless the rate of expansion for cement and aggregate is exactly the same stresses will be set up which could lead to cracking. In the situation being considered, where the cement shrinks and the aggregate expands, failure would certainly occur due to cracking of the cement. There is however another important property of concrete - it is porous. A large number of the cavities are adjacent to the particles of aggregate, and some shrinkage of paste can take place without greatly affecting the strength of the concrete \(^6\). However this only delays the initiation of cracks and the subsequent loss of strength. Surface crazing and very fine cracking within the body of the concrete begin at around 300°C, while deeper cracking takes place as the temperature increases to 500-550°C. By which time increased shrinkage of cement paste due to the dehydration of the calcium hydrate has occurred and the aggregate is also expanding rapidly.

1.2.1.1.4 Thermal movement: It is the net result of the cement paste shrinkage and the aggregate expansion that is considered in this context and since loading has an effect both the load and no load conditions are examined.

Fig.2\(^7\) shows the no load case. Upon initial heating expansion occurs, but for temperatures of up to 300°C the cracking that takes place is not great. On cooling the aggregate returns to approximately its initial size but the cement paste shrinkage is not recoverable, the net result is therefore a shrinkage. However when the concrete is heated to higher temperatures substantial cracking occurs.
This cracking reduces the restraint contributed by the cement paste shrinkage on the aggregate expansion. On cooling because much of the cracking is irreversible the net result is a residual expansion.

The situation is further complicated when loading is introduced during the heating cycle. Fig. 3(7) gives an example of conditions under load. The applied load opposes the thermal expansion and so inhibits both it and the cracking. As the cracking is less the shrinkage of the cement paste has a greater effect on the overall thermal movement partly due to it opposing the aggregate expansion. This time creep is also present adding to the tendency towards contraction. The overall effect can be to cause contraction at relatively low temperatures. Another factor which influences this situation is the rate of temperature rise. It does so by its effect on the creep of concrete.

It can be appreciated that the situation is complex and although these relationships can be identified, no acceptable theories have been produced to enable thermal movement and cracking at high temperatures to be predicted with any confidence(7).

1.2.1.1.5 Compressive strength(8,9):- Fig. 4(8) gives the relationship for compressive strength against temperature. For various considered situations the curve is of the same general shape although the actual values vary as follows:-

(a) The residual strength (i.e. strength on cooling) is lower than the strength at elevated temperature. This is considered to be due to increased cracking during cooling.

(b) Restrained concrete shows greater strength than unrestrained concrete, this is thought to be due to the applied stress opposing the development of cracks.
(c) For a given temperature lean mixes lose less strength than richer ones, although rich mixes have a higher original strength.

(d) Water/cement ratio has little or no effect on the strength-temperature relationship, although lower w/c ratios have a higher original strength.

The general curve shows that strength decreases with temperature although occasionally an increase in strength has been reported (cf. section 2.2.2.2 of this work). It would appear that the curing and moisture conditions of the concrete at the start of the test are very important (7). At 400°C and above deterioration becomes rapid. It is generally considered that concrete subjected to temperatures of up to 300°C will have sufficient strength remaining to be re-used. It will be remembered that fire cracking begins to appear at about 300°C. At 400°C and above the dehydration of the calcium hydrate causes rapid cement paste shrinkage and at 575°C the quartz transformation occurs and so the cracking increases. From this it can be appreciated that the cracking in the concrete affects the compressive strength, as would be expected.

1.2.1.1.6 Stress-strain curve:- The stress-strain curve for concrete in uniaxial compression at high temperatures shows a reduced peak stress with an increase in the strain range compared with ambient conditions. There is, also, an increase in the strain at peak stress (7).

1.2.1.1.7 Modulus of elasticity:- A notable reduction in Young's modulus occurs when concrete is heated. Fig. 5 (7) gives results for specimens heated to temperature under no applied load. There are similarities between the behaviour of Young's modulus and the compressive strength in that the application of a load during heating restricts the loss of stiffness. Similarly, as with strength, a
reduction of stiffness then occurs on cooling. Therefore it can be suggested that compressive loading restrains the cracking caused by thermal incompatibility and so slows down the reduction of stiffness on heating.

1.2.1.1.8 Creep: Creep is greatly increased when concrete is exposed to high temperatures as Fig. 6(7) indicates. The results are for specimens that were brought up to and maintained at a constant temperature before testing. It should be noted that while at ambient temperature creep can be considered as being a linear function of stress, at elevated temperatures this no longer holds true(7).

1.2.1.1.9 Relationships between thermal expansion, shrinkage, creep, cracking and strength: It has been seen that the thermal incompatibility of the aggregate and cement paste causes cracking which in turn leads to a loss of strength and stiffness. Put in another way the stress-strain curve changes so that the peak stress is reduced while the strain is increased both before and after the peak stress is reached. This loss of stiffness enables a greater stress redistribution to take place than is possible in the cool state. In addition creep is increased at elevated temperatures enabling the relaxation of thermal stresses to occur. These effects build up causing concrete at elevated temperatures to be a more ductile material than at ambient conditions. Over simplification must be avoided since the ductility is dependent on the small scale cracking and, as noted previously, this can be affected by the loading under consideration. However generally speaking considerable thermal strains can be withstood by the concrete at elevated temperatures. The comments above on the stress-strain, modulus of elasticity and creep characteristics are based on results for specimens that have been heated to the required temperature and then have been tested. In actual fire conditions the situation is different as load is being continuously applied throughout the heating up period. Thus
in an actual situation the problem becomes quite complex. It can perhaps be summed up by the following paragraph from ref. 7.

'The effect of loading on the strain during heating is not merely additive to the thermal expansion as is assumed in linear elasticity and simple structural theories. A change in load modifies the potential for cracking, affects the stiffness at a particular temperature level and so helps to determine the manner in which the paste and aggregate work together.'

1.2.1.2 Steel

1.2.1.2.1 Loss of strength:- The loss of strength is significant while the steel is hot, but except for cold worked steel, the recovery on cooling from temperatures up to 700°C is practically complete (10,11) (see Fig. 7(11)). The temperature at which steel begins to give an unsatisfactory performance in a fire depends on the applied stress at any point. However for bars with a maximum design load to carry there is a 0.87 safety factor. From Fig. 7 a drop in strength of 87% occurs at approximately 400°C. This can be taken to be the basic maximum temperature the steel can sustain adequately and adjustments must be made for smaller applied stresses. If the steel has withstand the fire then due to recovery of strength on cooling it will be of some use afterwards. The reason for cold worked steel not recovering as well is discussed in more detail in 4.2.2.

1.2.1.2.2 Relaxation losses:- Prestressing steel is often the most seriously affected reinforcing material due to the loss of tension caused by relaxation effects when creep occurs. In this case it is necessary to establish the temperature reached in order to estimate the likely damage. To emphasise this it can be noted that the strength is likely to be reduced by more than 50% at temperatures of approximately 400°C (11).
1.2.1.3 Summary

From this study of the effect of heat on concrete and steel the following main points can be noted.

(a) Concrete subjected to temperatures up to 300°C is capable of being reinstated.

(b) The thermal movement within the concrete causes cracking to occur as the temperature increases. This cracking reduces both the strength and the stiffness of the material producing a slightly more ductile material than at ambient conditions.

(c) The lower grade or mild reinforcing steels recover most of their strength after cooling from temperatures up to 700°C, whereas such recovery in cold worked higher grade steel is not so good.

(d) Prestressing steel must be carefully examined due to possible loss of face in the tendons caused by relaxation during heating.

1.2.2 Effect on Individual Members

Here it is necessary to look at the effect of the interaction between the concrete and the reinforcing steel on the member.

1.2.2.1 Cracking:— The reinforcing steel within the concrete helps to propagate both parallel and perpendicular cracks. This is caused by the variation in the different rate of expansion of steel to concrete. In reinforced concrete design it is considered that concrete and steel have nominally the same coefficient of linear expansion. This assumption is for the normal range of structural performance for temperatures between -20°C and +50°C. However this assumption is not justified at higher
temperatures (see Fig. 34 and ref. 47) where differences in the thermal strains of steel and concrete cause an internal restraint to occur. As the steel expands tensile stresses can be set up within the concrete and so cause cracking perpendicular to the concrete face or soffit. Also the movement of the steel can cause that plane to become a plane of weakness enabling cracking parallel to the concrete face or soffit to occur more easily, hence spalling can take place in the form of the cover to the steel sloughing off.

1.2.2.2 Deflection:– This results from a loss in steel strength at higher temperatures. Another result of this loss of strength is reinforcement buckling which can happen where spalling occurs leaving a length of bar exposed to the fire. This in turn can cause tensile elongation of the shear reinforcement which should consequently be checked for damage.

1.2.2.3 Bond:– To give a satisfactory structural performance reinforced concrete depends on the bond between the steel and the concrete, and so the effect of temperature on the bond is of great importance. If the concrete and steel both withstand the given conditions but the bond is destroyed the member will not fulfil its structural requirements. Current literature is remarkably silent on the residual bond strength following heat treatment. However, tests have been reported which tend to suggest that the relative decrease in bond due to temperature is slightly greater than that of concrete compressive strength. This aspect will be considered in more detail later.

1.2.2.4 Failure of a member:– Consider a simply supported beam under uniform loading. The maximum bending moment is at mid-span. When the beam is subjected to elevated temperatures the steel heats up and so begins to lose its strength. When the steel strength is so reduced that the ultimate bending moment capacity (M_u) is
1.2.3 Effect on the Overall Structure

Although a concrete structure is made up of different members they come together to form a monolithic type construction. This results in the structure acting as a composite whole instead of a collection of individual elements. In its response to fire there are two main points to note.

(a) The ability of reinforced concrete to redistribute forces and bending moments within a continuous structure can increase the fire resistance of a particular member. For example consider a continuous beam under uniform load. As the steel temperature increases the ultimate bending moment capacity at mid-span can decrease to zero without failure occurring provided that the supports can cope with the redistributed bending moment. For this to work satisfactorily sufficient top reinforcement must be provided over the support. This can increase greatly the fire resistance of the member compared with the example given in 1.2.2.4.

(b) An increase in fire resistance also can occur due to restraint being given to a particular member by the surrounding structure. If restraint is available a pre-stressing effect can be set up, as the member tries to expand due to the heat. However this effect will not continue after cooling has taken place, and it will only occur when the fire is localised and does not reach the members that are providing the restraint. Movements caused by fire can create distortion in the structure as a whole and so alignment should be checked. Also expansion can cause damage to those parts not directly affected by the fire.

Due to the good fire resistance properties of concrete the structure usually withstands collapse and also retains
sufficient structural integrity to enable it to be reinstated.

1.3 REINSTATEMENT

1.3.1 Evaluating the Damage

Concrete heated to beyond 300°C is not suitable for reinstatement and fires can reach temperatures of 1000°C or more. However the outside layers of concrete insulate the interior and so the temperature gradient within a member is steep. This is particularly so for short duration fires. To add to this the dehydrated outer layers have an even better insulation property than the normal concrete, keeping the interior considerably cooler than the outside.

The temperature gradient and resultant damage to each individual member has to be evaluated when faced with the choice whether or not to reinstate. The ideal method of evaluating the fire damage would be to accurately assess the temperature attained, and from this to predict the residual strength. However the estimation of the temperature reached is not so straightforward and it is common practice to combine it with tests for the residual strength of the steel and concrete. All the results are then compared to ensure consistency.

An estimate of the temperature reached during the fire can be made by an examination of the debris. Different materials react in certain ways and melt at different temperatures. Only the surface of the concrete will be exposed to the fire. The temperature of the interior depends not only on the temperature of the fire but also on its duration. One of the main methods of deducing the temperature the concrete has reached is the colour change that takes place. The degree of colour change, shown in Table 1 (13), varies for different types of concrete, at a given temperature, but most concretes in general show this
characteristic to some extent. The pink colour at 300°C is the most useful characteristic, that being the maximum temperature allowable for reinstatement purposes.

Colour change is not the only way of estimating temperature. Surface cracking, deeper cracking, spalling from steel, quartz popout and general weakening and friability of the concrete also help in this respect (Table 1[13]).

Other methods of evaluating the fire damage include the calculation of fire severity and comparison with fire test data[14], concrete strength tests on cores, steel tensile tests, measurement of steel strength by metallographic methods[15], ultra-sonic pulse testing[16] and load tests. A very recent development in this area is the examination of the thermoluminescence of concrete particles taken from small cores at various depths in a component to establish the temperature and thermal gradients attained during a fire[17]. The possible application of acoustic emission in this field led to a small number of tests being carried out in this area which are the subject of chapter 6.

1.3.2 Design and Methods of Reinstatement

The aim when reinstating a member is to restore it to a condition in which it will be able to carry the applied loading. In order to achieve this it is sometimes necessary to provide structural as opposed to cosmetic repair. The basic design procedure[11,18] is,

(a) assess the strength of the member remaining after the fire,

(b) compare the remaining strength with the actual strength required.
(c) if the residual strength is less than that needed, add reinforcement and concrete capable of sustaining the difference; if it is greater then a cosmetic repair only is required.

Having designed for the reinstatement various methods can be adopted to carry out the work. Gunite\(^{(11)}\) prepackaged concrete\(^{(19)}\) and the use of formwork\(^{(12)}\) are all feasible methods of approach depending upon the exact nature of the repair to be made.

1.4 DESIGNING FOR FIRE RESISTANCE

It has been seen that concrete is a good fire resistant material and how this leads to the possibility of reinstating a burnt out structure. However it is possible to reduce the damage to the building even further by considering fire endurance in the initial design. When designing for fire resistance there are two main approaches which complement each other. These are,

(a) careful appreciation of details in the design that can protect a structure from fire damage;

(b) designing the required fire resistance into structural members.

The first of these approaches has been discussed elsewhere\(^{(3,20)}\), and also the second has received some detailed examination\(^{(18,20-22)}\). At the moment in relation to the second approach the code (CP110: Part 1:1972\(^{(23)}\)) gives a guide to fire resistance in the form of minimum values of the width or depth of the member and the cover to the reinforcing steel. It is an arbitrary method based on fire test results and does not take into consideration the individual circumstances involved. However in many cases a satisfactory solution is obtained.
A different approach to the problem is to design the required fire resistance into the member by analytical means based on known heat transfer and material property behaviour. This is termed 'rational design'. Briefly stated it involves obtaining the temperature distribution throughout the member section, then, knowing the effect of temperature on the material properties, estimating the structural strength for the given conditions. The required fire resistance can be obtained by varying the loading, amount of steel reinforcement, concrete cover to reinforcement, and type of concrete used. The type of member is of importance - continuous beams and slabs giving better fire resistance than simply supported ones under the same conditions. Also the ability to restrain thermal expansion affects the design. In addition to this it is essential that there is adequate detailing of the reinforcement.

However, little attention has been paid to the quality of bond between the concrete and steel in fire resistant design. Therefore it would seem desirable to examine the influence of high temperature on the bond between the materials in order to better understand the performance of structures both during and subsequent to a fire and to make a better assessment of the design approach. This matter is examined in more detail in subsequent chapters of this work.
CHAPTER 2

BOND IN REINFORCED CONCRETE AT AMBIENT
AND ELEVATED TEMPERATURES

2.1 BOND AT AMBIENT TEMPERATURES.

2.1.1 General

Bond between the steel and concrete is an essential part of reinforced concrete. Without it the various members, such as beams and slabs would not be able to provide a satisfactory structural performance. When the steel and concrete are bonded together the reinforcing prevents the concrete from failing prematurely by resisting the majority of the tensile force and so allowing the compressive strength of the concrete to be utilised to the full. If there is no bond between the steel and concrete then reinforced concrete loses its integrity as a composite material.

2.1.2 Nature and Mode of Failure of Bond

The nature and mode of failure of bond differs between plain and deformed bars.

2.1.2.1 Plain Bars

In this case the bond is made up of a combination of adhesion, shrinkage of the concrete causing frictional resistance, and the surface characteristics of the bar. Mikhailov, as reported by Watstein and Bresler (24), studied the relative importance of the adhesion and frictional bond. He noted that the contribution to the bond in terms of stress made by adhesion between the steel and concrete amounted to only 0.5 to 0.7 N/mm² which is not of any great significance. However the adhesion plus frictional resistance resulting from shrinkage contributed 25 to 30% to the bond resistance.
while the surface roughness of the bar and any change in its lateral dimension throughout the considered length accounted for 70 to 75% of the total bond strength. Plain bars can fail in bond by being pulled through leaving the surrounding concrete intact. Alternatively splitting of the concrete can take place but only after slip has occurred at both the loaded and unloaded ends of the bar.

2.1.2.2 Deformed Bars

In the case of deformed bars, although frictional resistance and adhesion are still present, the major factor in the development of bond resistance is the bearing of the deformations against the concrete and the ability of the concrete to withstand this pressure. Due to the change in the nature of the bond resistance there is also a difference in the failure mode between plain and deformed bars. Failure with deformed bars is nearly always associated with longitudinal splitting through the concrete which provides the smallest cover to the bar. This splitting often takes place before the unloaded end of the bar begins to slip.

Further insight into the nature of bond strength has been gained from studies of internal cracking caused by the development of bond stresses in concrete tension prisms. Broms and Lutz (25) reported that radial cracking takes place starting at the steel-concrete interface and stopping before it reaches the concrete surface. Bresler and Bertero (26) showed that for low loading levels principal tensile stresses at the bond interface are inclined at an angle with the longitudinal axis varying between two concrete cracks from about 60° at the crack face and decreasing to 0° at the midpoint (Fig. 8(a)). Experimental investigations by Goto (27) also showed that inclined cracks developed within the concrete prism, and that these cracks were approximately normal to the direction of the principal stresses being inclined at an angle of 45-80° to the bar axis (Fig. 8(b)).
Based on these studies the mechanism of bond development with increasing loads has been described as follows by Watstein and Bresler (24).

"At low stress levels in the steel reinforcement high principal stresses occur only at the interface, in the zones adjacent to the full transverse cracks where local inclined cracking occurs. This local cracking relieves the high tension near the ends and shifts the zone of maximum principal stress inward from the transverse crack face. With increasing load these maximum stresses reach the tensile strength of the concrete generating additional internal cracking. With further increase in load the process of internal cracking and consequently of shifting the peak stresses continues. At yield stress level the internal cracks propagate through most of the prism, so that concrete adjacent to the steel reinforcement forms a 'boundary layer' of teeth-like segments which resist the pullout forces by wedging action, as shown in Fig. 9.'

The deformation characteristics of this boundary layer, as yet, are not defined quantitatively in a realistic manner.

2.1.3 Anchorage and Flexural Bond Stress

Two types of bond stress can be identified. First there are the anchorage bond stresses which occur at the end of the steel bar whether extended into a support or cut off within a span, and then there are the flexural or local bond stresses which generally vary with the shear or change in bending moment.

The anchorage bond stress is taken to be

$$\sigma_b = \frac{P}{\Sigma u l_v} \quad (1)$$

where

- $\sigma_b$ = bond stress
- $P$ = force in bar
- $\Sigma u$ = effective sum of bar perimeters
- $l_v$ = effective embedment length

and where the bond stress is assumed to be constant over
length $l$. Equation 1 is used to calculate the average bond stress obtained in pull-out tests.

The flexural or local bond stress is due to the change in steel force along the length of the bar and is given by

$$
\sigma_b = \frac{\delta P}{\Sigma u.5x} = \frac{V}{\Sigma u.z}
$$

where $\delta P =$ change in tensile bar force over an elemental length, $5x$.

$V =$ shear force

$z =$ lever arm

In this case there are a number of assumptions,

(a) the concrete does not resist tension,

(b) the tension in the steel varies directly as the ordinate of the moment curve throughout the beam length, and

(c) the beam is not cracked.

Eqn. 2 is used to calculate the average bond stress over length $5x$ in beam tests.

2.1.4 Development of Bond Tests

When dealing with bond quality it is not just the maximum bond strength available that is important but the bond stress in relation to the amount of slip between the steel and concrete. The most useful characteristic is the local bond resistance to slip since from this relationship attempts can be made to establish crack widths and spacing in loaded reinforced concrete members.

In some early reports of bond tests only the maximum bond values were given. Then Abrams, as reported in Watstein and Bresler (24), conducted a series of tests using
pull-out and beam specimens measuring both slip and bond strength. Since then many different experiments have been undertaken mainly along the lines of either pull-out or beam tests (28). Slip between the steel and concrete was measured at the loaded and unloaded ends of the specimen while the average bond stress was calculated for a specific length of bar in accord with the definition for pull-out or beam tests mentioned above i.e., equations 1 and 2. This enabled the behaviour in the anchorage zone to be investigated and different bar types to be compared.

2.1.4.1 Bond Stress Distribution

However the assumptions made in 2.1.3 above are not strictly applicable to conditions in practice. The bond stress over a length of bar is not constant, a peak being reached near the loaded face of pull-out specimens and close to flexural cracks in the case of beams. The assumption that concrete does not resist tension and that the beam is not cracked are contradictory. The concrete does resist tension until its tensile strength is exceeded causing a crack to occur. Hence the tension in the steel does not vary directly as the ordinate of the moment curve throughout the beam length. Therefore tests were carried out to study the bond stress distribution. This is achieved by measuring the steel strain, thus giving the steel stress, at different points along a bar as equation 3 makes clear.

\[ \sigma_b = \frac{\delta P}{\Sigma u \delta x} = \frac{A_s \delta \sigma}{\Sigma u \delta x} = \frac{A_s E_s \delta E_s}{\Sigma u \delta x} \]  

Equation 3

Various methods have been used to measure the strain in the steel. Watstein (29) provided holes in the concrete giving access to the steel bar, thus enabling strain to be measured (see Fig. 10(a)). Electrical resistance strain gauges have been fixed to the outside of the bar while Mains (30) placed them within the steel reinforcement. To do this the bar was cut longitudinally, a groove milled out enabling the gauge to be attached, with the two halves of the
bar being tacked welded together again (see Fig. 10(b)).

All these methods have disadvantages. The access holes and strain gauges attached to the outside of the steel reinforcement disturb the conditions at the bond interface, while the gauges placed within the bar cause its cross-sectional area to be reduced hence increasing the steel strain for a given load.

2.1.4.2 Bond Slip

It is even more difficult to measure the internal slip as in this case the concrete strain has to be considered.

A radiographic method was used by Evans and Williams\(^{(31)}\) where platinum whiskers which were embedded in holes drilled in the steel bar protruded into the concrete. X-ray photographs were taken before and after loading showing the relative displacement of the sheared platinum markers. An approach by Moore (reviewed by Nilson\(^{(32)}\)) was to use an air gauge to measure the flow of air being regulated by the displacement between the steel and concrete. Both of these techniques have drawbacks, the former placing restrictions on the specimen size and being expensive while the latter shows signs of lacking accuracy and sensitivity. Further methods of measuring slip were also summarised by Nilson\(^{(32)}\). Ruiz utilised electrical resistance strain gauges to develop a slip gauge. One end of the slip gauge was bonded to a mortar block and the other end to the steel bar while the central portion was left unbonded. The mortar block was embedded in the concrete so that the gauge was parallel to the bar axis. To measure slip directly at internal locations Wahla used miniaturised differential transducer gauges. However the great disadvantage of methods where gauges are present at the bond surface is that they will affect the bond distribution besides requiring great care in the preparation of the specimen. This led Nilson\(^{(32)}\) to place internal concrete strain gauges within the concrete mass close to but
not on the embedded bar in addition to strain gauges within the steel bar (Fig. 11). The concrete strains were not measured at the bar surface introducing a slight error. Also a great number of gauges are required to obtain a continuous distribution along the length of the bar.

2.1.4.3 Bond-Slip Relation

A number of investigators have attempted to measure the local bond-slip properties by testing a small bond length. Rehm (33) employed this method by using 100 and 200mm cubes and testing a differential bond length of, for example, 16mm (Fig. 12(a)). By doing this the stress-slip curve could be arrived at directly from the load applied and the measured end slip. This simplifies the experimental procedure considerably although great care must be taken in producing the specimens because of the small bond length.

Other investigations along this line have been done by Stocker and Sozen, as reviewed by Yannopoulos (34). They used a similar procedure, with concrete specimens of 100x230mm cross-section and 200mm long. They tested four different embedment lengths 12.7, 25.4, 38.1 and 50.8mm when studying the bond strength of strand, and concluded that in this case the shortest length giving reliable and consistent test results was 25.4mm. Edwards and Yannopoulos (34,35) used a slightly different kind of specimen (see Fig. 12(b)). A bond length of 38mm was adopted with concrete covers of 35 and 25mm.

These specimens also have disadvantages when compared to conditions in practice. The Rehm and Stocker/Sozen type of specimen gives greater lateral restraint than normal whilst that used by Edwards and Yannopoulos represents conditions more at the beam end or crack face only. Whatever form of specimen is used disadvantages are apparent in one way or another. However the 'differential bond length' type gives the desired results directly and also makes testing both
cheaper and easier.

2.2 BOND AT ELEVATED TEMPERATURE

2.2.1 Review of Previous Work

Previous work on bond strength both at elevated temperatures and in the residual (i.e. cooled) state is summarised in Table 2 and Fig. 13, much of which has been described in some detail earlier (see appendix 1).

2.2.2 Comments on Previous Bond Work at Elevated Temperatures

2.2.2.1 Experimental Approach

2.2.2.1.1 Test Specimen: It is noticable that they are all variations of the pull-out type specimen. Beam tests for bond at elevated temperatures do not appear to have been undertaken probably due to a desire to keep the tests both simple and inexpensive. The specimens vary from those with a long bond length (36-42) to those with a smaller embedment length where the end slip gives more of an indication of the breakdown in local bond (43-47), although even in these cases the lengths of 80 and 115mm are still too large to obtain an accurate assessment of the local bond stress-slip relationship.

The shape of the specimen adopted by Hertz (43,44) is of a different form than the others, seeking to make use of the way the compressive forces are transmitted at an angle of 45° to the bar axis (by using this method Hertz's aim to eliminate the possibility of failure by longitudinal cracking is furthered). In the test procedure 3mm neoprene 60° shore was placed between the concrete specimen and the steel seat of the test apparatus to reduce friction. However it is inevitable that there will be some friction still present that will cause a slight discrepancy in the transmission of the
compressive forces.

2.2.2.1.2 Rate of Temperature Rise: There are two approaches to this point of experimental procedure. The temperature can be increased so that it follows the standard fire test curve \(^{(48)}\). This gives a rapid rate of temperature rise simulating conditions in an actual fire. By using this approach (e.g., Reichel \(^{(42)}\)) the results can be compared to other fire test data. Alternatively a much slower rate of temperature rise can be adopted (e.g., Harada et al. \(^{(39)}\)). In this case a gradual rise in temperature throughout the specimen is obtained preventing large temperature gradients from occurring over the section. The result is that the mechanical stresses, caused by the difference in the thermal expansion between the outside and the centre of the concrete, are eliminated. Hence the effect on the bond caused by temperature alone can be seen more clearly.

2.2.2.2 Compressive Strength of Concrete

A number of the investigations mentioned carried out tests to obtain the compressive strength of the concrete as well as the bond strength for the conditions under consideration. The various results are given in Fig. 14 and can be seen to vary considerably. The results of Harada et al. \(^{(39)}\) and Kasami et al. \(^{(40)}\) show the expected decrease as the temperature increases. Significant increases in strength for the initial temperature increase are given by both Ghahramani/Sabzevari \(^{(38)}\) and Milovanov/Pryadko \(^{(36)}\), while the work done in Edinburgh \(^{(41)}\) shows the strength at 500°C to be almost equivalent to that at ambient temperature. To some extent the variations in the values can be explained by the difference in the cement and aggregate and the size and shape of the specimen adopted e.g., Milovanov and Pryadko \(^{(36)}\) used refractory aggregate and water glass cement. Harada et al. \(^{(39)}\) used cylinders whereas Ghahramani and Sabzevari \(^{(38)}\) adopted cubes.
2.2.2.3 Bond Strength

The residual bond strength results have been plotted in Fig. 15 for plain bars, and Fig. 16 for deformed bars. The results of Harada et al.\textsuperscript{[39]} and Kasami et al.\textsuperscript{[40]} have been plotted on both graphs as the bar types are unspecified.

Due to the differences in the experimental procedures it is not possible to make a direct comparison between the various curves. To compare the results the conditions over all the tests would need to be uniform, whereas in reality the differences are many - rate of heating, size and shape of specimen at test, concrete strength. Other factors that could cause variability in the different results and which have not been specifically mentioned as yet include the direction of casting which affects the extent of the voids at the concrete/steel interface, the surface condition of the reinforcement e.g., lightly corroded steel will give a better bond performance than either smooth or heavily rusted bar, the curing conditions which affects the concrete shrinkage and the concrete hoop stress surrounding a bar.

In view of this a wide scatter in the various curves is to be expected and helps to explain the contradictory nature of some of the results e.g. Reichel\textsuperscript{[42]} reports the plain bar giving a greater reduction in bond strength compared with the deformed bar whereas with Ghahramani and Sabzevari\textsuperscript{[38]} the situation is reversed. Fig. 17 shows the results for deformed bar at elevated temperatures from the work of Diederichs/Schneider\textsuperscript{[45,46]} and Sager/Rostasy\textsuperscript{[47]}. From this figure the different concrete strengths and aggregate types can be seen to affect the bond results.

2.2.2.4 Summary

The conclusions reached by the different investigations show that the reduction in bond due to an increase in temperature could be a critical factor when assessing the
strength of heat treated reinforced concrete members.

Harada et al.\(^{(39)}\) Morley\(^{(41)}\) and Ghahramani and Sabzevari\(^{(38)}\) reported the bond quality deteriorating more rapidly than the concrete compressive strength whilst Reichel\(^{(42)}\) concluded that the residual bond strength decreases at a greater rate than the residual steel strength. Hertz\(^{(43,44)}\) has shown that for temperatures up to 400\(^{\circ}\)C the ratio of bond strength/concrete strength is reasonably constant but decreases for temperatures above 400\(^{\circ}\)C while Milovanov and Pryadko\(^{(36)}\) state that for refractory concrete there is considerable bond strength up to 450\(^{\circ}\)C although they emphasise the need to consider the effect of bond when dealing with elevated temperatures.

Therefore a number of investigators have carried out work giving an indication of the effect of temperature on the bond strength, however due to the differences in experimental procedures comparisons are difficult. Also out of the many variables affecting the bond strength only a limited number of parameters have been considered e.g., two types of bar and concrete strength were looked at by Reichel\(^{(42)}\) while Ghahramani and Sabzevari\(^{(38)}\) used two different heating cycles. To add to this the variable nature of concrete as a material means that, even under uniform test conditions, there can be a wide scatter in the results. Therefore a considerable amount of test data is required to produce an accurate assessment of the bond strength-temperature relationship.

The more recent work of Sager and Rostasy\(^{(47)}\) looked at two different normal weight concrete strengths as well as light weight concrete using a total of five different aggregates, thus investigating the effect the type of concrete has on the bond performance in more detail than anything prior to it.

All this shows that the work carried out is only the
beginning of research into this particular area, and that this lack of information is causing more detailed study of the problem to take place e.g., ref. 47. It is with the purpose of adding to the understanding of this particular field, of the high temperature effects on the bond in reinforced concrete, that the work presented in the following chapters was undertaken.
CHAPTER 3

TEST APPARATUS AND EXPERIMENTAL PROCEDURE

3.1 TEST APPARATUS

3.1.1 Design Requirements

The aim of the experimental work is to investigate the effect of elevated temperatures on the bond strength between reinforcing steel and concrete during and after heating. When formulating the test procedure to achieve this aim, two main design requirements had to be taken into account.

(a) In practice structures are normally under load during fires, therefore, to ensure the tests are as practical as possible a working bond stress had to be applied to the specimens during the heating cycle.

(b) In 2.1.4 it was noted that the most useful result is the bond stress-slip relationship, and so, the slip had to be recorded throughout the duration of the test.

3.1.2 Test Specimen

One of the earliest decisions that had to be made was whether to use a beam or pull-out type test specimen. It can be argued that a beam test is more consistent with conditions found in practice, however, difficulties in obtaining the bond stress-slip result are apparent. Methods of obtaining bond stress, steel strain, and bond slip have been discussed in the previous chapter. Compared with the pull-out type specimens having a small or differential bond length, where the stress-slip relationship can be measured directly, the beam specimens are more expensive and require great care in their preparation at ambient temperatures. These drawbacks are magnified when considering elevated
temperatures e.g. high temperature strain gauging. Therefore it was decided to use a pullout type arrangement with a small embedment length, keeping the casting and testing procedure as simple and inexpensive as possible. This enables a greater number of tests to be performed with the hope of providing more positive information than would be the case otherwise. Another reason for choosing this approach is that of being able to concentrate on the bond failure, whereas, with beam specimens failure, although designed to be in bond, can be complicated by flexural or more likely shear effects. Eventually it will be necessary to know the bond performance and the various interactions within beams at elevated temperatures. However at this initial stage of investigation there is some advantage in isolating the bond as much as possible from the other criteria, as it enables basic bond data to be established which will hopefully help in understanding future tests undertaken in the flexural situation.

The test specimen chosen is shown in Fig. 18. A cylindrical shaped specimen is used as it is more appropriate for producing a uniform heat flow from the outside surface to the bond interface than for example a prism.

A bond length of 32mm was decided upon for a number of reasons. It was desirable that the bond length be greater than the maximum aggregate size of 19mm. Although Stocker and Sozen (34) (see section 2.1.4.3 of this work) were dealing with strand, for consistent results, their shortest bond length of 25.4mm is a warning against making the embedment length too small. On the other hand to aid the understanding of local bond characteristics as small a differential length as practical is desired. In view of these considerations a length of two bar diameters i.e., 32mm was chosen.

3.1.3 Test Furnace

A small electric furnace was built to heat and test four
specimens simultaneously. The furnace was designed for temperatures up to 1000°C which makes it necessary to apply the load and measure the slip from the outside of the heating chamber.

3.1.3.1 Application of Load

The steel reinforcing bar extended from the concrete specimen right through the furnace base allowing the load to be applied via a hydraulic jack and loading yoke system (see Fig. 19). The end of the steel bar was threaded with a \( \frac{1}{2} \) inch Whitworth thread enabling the loading yoke to be attached to the bar by a nut and bolt type assembly. The hydraulic jack was positioned on the loading yoke seat and pushed against the reaction beam, hence applying a tensile force to the steel reinforcing bar. A continuous record of the applied load was made possible by incorporating a load cell into the set-up. It was placed between the jack and the reaction beam and was bolted to the beam to keep it permanently fixed in position. This procedure was repeated for each of the four specimens capable of being held in the furnace.

3.1.3.2 Measurement of Slip

In order to obtain a continuous record of movement between the steel and concrete the procedure shown in Fig. 20 was used. A fused silica hollow cylinder was placed concentrically on top of the concrete specimen and fixed to its other end, by means of a cap arrangement, was a displacement transducer. A fused silica rod was placed between the steel reinforcing bar and the transducer plunger. This enabled slip to be measured between the steel and the concrete at the top of the specimen from outside the furnace chamber. Opaque vitreosil silica was used for the cylinder and rod. It consisted of silica fused into a pure homogeneous and uniform product with a very good high temperature performance, the coefficient of thermal expansion being
The transducers were Sangamo DC/DC linear displacement transducers. They were self-contained, each with built-in linear variable differential transformer (LVDT) type windings, oscillators, demodulators and filters and when used in conjunction with a stabilised D.C. power supply they produced a D.C. voltage output proportional to the physical stroke of the armature. Also shown on Fig. 20 are the thermocouple wires. These come from the specimen, travel up the cylinder and out through holes provided in the transducer cap.

3.1.3.3 Furnace Heating Requirements

As well as designing for a maximum temperature (i.e. 1000°C) it was necessary also to be able to control the rate of heating. A linear rate of temperature rise was desired which could be reduced to zero allowing the furnace to remain at a constant temperature for a set period of time. In addition to this the horizontal and vertical temperature distribution through the heating chamber had to be as uniform as practically possible.

A Eurotherm temperature controller type 027 was used in conjunction with a Eurotherm temperature programmer type 125 to provide a linear rate of temperature increase. The optimum switching mechanism to use with this type of arrangement is by thyristors, however, in this case to keep costs to a minimum the simpler contactor type switch was utilised. This is illustrated in Fig. 21(a). It enabled the contactor to turn the current on and off increasing or decreasing the rate of heating as required. The result, however was a stepped increase in temperature rise as opposed to the smooth linear rate desired. In order to overcome this an Electrothermal energy regulator type MC233 was incorporated into the system so that the full load could be reduced to an appropriate level.
As Fig. 21(a) shows the furnace had 12 heating elements. These 12 elements were divided into three zones, each zone corresponding to one phase of the 3 phase mains supply used. When the power was switched on by the main contactor switch each of the three zones could be controlled individually by the 3 phase energy regulator, thus helping to obtain a uniform vertical temperature distribution.

The heating elements themselves were made of 1.219mm diameter Kanthal DSD wire. Each element consisted of 12.58m of wire wound round a Morgan Triangle impervious aluminous porcelain supporting rod of diameter 17.5mm, and had a stretch ratio of 3.5:1. The resistance per element was 14.55 ohms giving an output of 0.99kW. Four elements were in each zone and these were connected as shown in Fig. 21(b). The two elements positioned on the same side of the furnace were connected in series and the two pairs were connected in parallel. For maximum heating over the 3 phases a current of 49.5A was required giving a total output of 11.88kW.

3.1.3.4 Furnace Construction

Fig. 22 shows the heating chamber which had dimensions of 1000x250x300mm high with base, side wall, end wall and lid thicknesses of 200, 200, 175 and 175mm respectively. The most substantial part of the furnace is the base, as it has to resist the pressure from the concrete specimens when they are under load. Adequate compressive strength was provided by forming the base from Morgan's Tri-mor extra high strength castable refractory material. Four 25mm holes were provided for the steel bars to pass through the base to the outside of the furnace.

The governing factor in the construction of the side walls was the need to support the heating elements. It was decided to do this by using blocks made from Morgans Tri-mor insulcast insulating castable refractory material, not as
The furnace lid had to be as light as possible for easy man handling. It was made also from Triton Kaowool and mineral wool held together in a steel casing. As can be seen from Fig. 23 the lid was in three sections. Not only did this facilitate easy lifting, it enabled the two outer portions to be removed, allowing the furnace to cool, without the centre section and hence the displacement transducers being disturbed. In the centre section there were four 60mm holes for the placement of the fused silica cylinders, mentioned in 3.1.3.2., along with the furnace thermocouples.

All the electrical connections were made at one end of the furnace where a cover was provided for this purpose (see Fig. 23).

3.1.3.5 Furnace Support

Fig. 24 shows the brick wall support for the furnace. As well as the walls there were the four support beams consisting of two steel channels each. The two channels of each beam were placed one on either side of the corresponding 25mm hole in the furnace base ensuring that when load was applied to the specimens the base was subjected to compressive stress only. Four steel I-sections acting as the reaction beams (see 3.1.3.1) were secured in the brick wall support as depicted in Fig. 24. An overall view of the test furnace is given in Fig. 25.
3.1.4 Recording Instruments

There were three sets of data to be recorded. Firstly there was the applied load which was obtained for each specimen via a load cell connected up to a 6 channel continuous trace Rickadenki chart recorder. Secondly the slip had to be monitored. The signal from a displacement transducer passed to a Sangamo signal conditioning unit and thence to another Rickadenki 6 channel chart recorder. Finally the temperature of the bond interface was obtained from thermocouples cast into the specimens and connected to a 24 channel Texas temperature recorder.

3.2 EXPERIMENTAL PROCEDURE.

3.2.1 Casting

Suitable moulds had to be made for casting the specimens. The mould base plates were made from steel and, to keep the cost to a minimum, rigid plastic tubing was utilised for the cylinder walls. Removal of the concrete from the moulds was assisted by a very slight taper on the tubing. During casting a mould was placed on a 'casting table' to which the steel reinforcing bar was fixed by means of a nut and bolt type arrangement (see Fig. 26). The bar was secured in this way to ensure that it remained perpendicular to and central with the mould base plate. One consequence of this was that the bar was cast in the direction it was eventually pulled. Soft plastic tubing was placed over the bar to prevent bonding from taking place over its entire length. In this way a 32mm bond length was obtained, the tubing being removed during the demoulding of the specimen. The temperature at the bond interface was measured by a thermocouple positioned in a specimen as shown in Fig. 18. It was placed just outside the bond length to reduce any effects its presence might make on the results. As Fig. 26 indicates four bond specimens were cast at a time along with 4 x 100mm test cubes. The concrete was compacted using a tamping rod and cured by
placing the cubes and specimens in polythene bags and storing in the laboratory for three months at a humidity of 60%.

3.2.2 Concrete Mix

Ordinary Portland cement was used along with a natural gravel aggregate of maximum size 19mm (shrinkage value\(^{\text{51}}\) = 0.07\%) to produce a 1:2.08:3.65 mix by weight with a water/cement ratio of 0.6. This gave a cube crushing strength of 35.0N/mm\(^2\) at an age of three months. The course and fine aggregates conformed to the sieve analysis limits of BS882:Part 3:1973\(^{\text{52}}\), the fines being consistent with a zone 3 sand.

3.2.3 Test Routine

3.2.3.1 Bond Test Specimens

The concrete was allowed to develop the majority of its strength by curing the specimens for three months prior to testing. Once the bond specimens were installed in the furnace the testing procedure was as follows:

(a) Apply load to those specimens to be tested under stress.
(b) Start the heating cycle at a rate of 2\(^\circ\)C/min.
(c) After the maximum temperature had been held constant for 1 hr. load to failure any specimens to be tested while hot.
(d) Switch off furnace and remove the outside portions of the lid for air cooling.
(e) 24 hrs later load the residual specimens to failure.

At the end of a test the specimens were removed from the furnace, the type of failure was noted along with the crack pattern, if any, and then the specimens were split by the indirect tension test method. Splitting the bond specimens
enabled the bond interface to be checked and also gave an indication of the concrete tensile strength.

The four 100mm cubes were crushed at ambient temperature to keep a check on the consistency of the concrete quality. Finally a tensile test was carried out on the steel bars to obtain their residual stress-strain curve. For the ambient temperature tests the procedure followed the same pattern with the exception of the heat treatment.

3.2.3.2 Concrete Strength Test Specimens

Cylindrical specimens used for obtaining the concrete compressive strength were cast using the same moulds as for the bond specimens. These cylinders were heated in the furnace and then removed for testing. Again the residual strength was obtained 24 hours after the 1 hour dwell at the maximum temperature.

3.3 TEMPERATURE DISTRIBUTION

A number of preliminary tests were carried out to ensure the furnace gave an adequate heating performance and also to determine the thermal gradient within the test specimen. Three different tests were performed. The first was to find the effect of positioning the specimen in the middle or at the end of the furnace while the other two established the radial and vertical temperature distributions within the specimen.

For the first case mentioned specimens were placed in the furnace with the thermocouple positioned as in Fig. 18 and the temperature was increased at 2°C/min. At 500°C there was a difference of 15°C between those at the middle and end. This increased to 20°C at a temperature of 750°C. The effect of this difference is negligible compared to the natural scatter in the bond performance, a conclusion, which, is verified by a study of the results
When determining the radial temperature distribution, thermocouples were placed at varying depths throughout the specimen. These were at levels just below and above the bond length as well as at the interface level itself (see Fig. 27(a)). The resulting temperature gradients for a rate of rise in temperature of 2°C/min are shown in Fig. 27(b). For the inner core of the specimen the temperature gradients were on average 0.77°C/min. This was slightly higher than the 0.6-0.7°C/min reported by Schneider and Diederichs(4,5) and felt to have no effect upon the concrete's thermal behaviour. The one exception to this was the t = 360min curve where the gradient over the inner 34mm increased to 0.88°C/min which was a little higher but only affects tests at furnace temperatures of 750°C and above. As would be expected the outer half of the specimen had a greater thermal gradient than the inner while the furnace temperature increased. However any disturbance that this caused was remote from the immediate vicinity of the bond interface.

The final temperature distribution test was to establish the vertical thermal gradient over the height of a specimen (see Fig. 28(a)). As indicated in Fig. 28(b) the distribution is reasonably uniform over the bond length and the upper half of the specimen. The temperature decreased towards the bottom of the furnace as, even with the three zone system in operation, the furnace was cooler at this point. There was a slight drop also in temperature at the very top due to the heat loss that took place through the various holes and slight openings between the three lid sections. However the most important requirement was satisfied, that of a uniform vertical temperature distribution over the embedment length.
CHAPTER 4

EXPERIMENTAL INVESTIGATION

MATERIAL CONTROL AND BASIC BOND TESTS

4.1 TEST PROGRAMME

4.1.1 Test Parameters

In this work the main parameter is the temperature. Six furnace temperature levels are examined, namely, 20, 150, 300, 450, 600 and 750°C. The latter being the highest temperature with which it was possible to complete the heating-up period within a working day.

There are many other variables that affect the bond performance and the following were selected for examination:

Four test conditions:
(a) stressed during heating; tested while hot
(b) unstressed during heating; tested while hot
(c) stressed during heating; tested when cool
(d) unstressed during heating; tested when cool.

Four covers: 25, 32, 46 and 55mm
Two bar types: plain and deformed
Three steady state bond stresses: zero, 2.45 and 3.70 N/mm²
Load cycling: 20 cycles between 1.0 and 3.70 N/mm²

As it was not possible to consider all the variables the following parameters were kept constant throughout the test programme:

Rate of heating: 2°C/min
Dwell at maximum temperature: 1 hour
Concrete mix: (see section 3.2.2)
Bar size: 16mm diameter
Embedment length: 32mm
Concrete was cast in the direction of pull

4.1.2 Sequence of Testing

A number of test series were undertaken to investigate the parameters above. In the following a batch is taken to be a set of four specimens cast, heated and tested together, while a series denotes a number of batches, one tested at each temperature level (see Table 3).

4.1.2.1 Test Condition:- As four test conditions ((a)-(d) in 4.1.1 above) were chosen and for each batch four specimens were made, it was possible to examine each test condition once per batch. Six batches were required to cover the six temperature levels considered. This process was repeated five times (series AV-AZ) giving five results for each test condition at each temperature. The advantage of using this method was that variations in the results due to any slight differences in the concrete mixes and test environments were averaged out more effectively. All of these tests were done for deformed bars and a cover of 55mm. For test conditions (a) and (c) a bond stress of 3.70 N/mm² was applied while the specimens for conditions (b) and (d) were unloaded during heating.

4.1.2.2 Cover:- Three series of tests (BX-BZ) were carried out for the three smaller covers i.e. 25, 32 and 46mm. Deformed bar and test condition (c) (i.e., steady state bond stress applied during heating; tested to failure when cooled) were used throughout. The relevant results i.e., those for test condition (c), from the AV-AZ series were used to compare the 55mm cover with the smaller values.

4.1.2.3 Bond Stress:- Two bond stresses (i.e., zero and 3.70 N/mm²) were taken into account in series AV to A2.
One more series, C, was completed with a bond stress of 2.45 N/mm²; deformed bar and 55mm cover being used.

4.1.2.4 Bar Type: - A series labelled D in Table 3, was tested using plain bars. All the specimens were examined under test condition (c) and had a cover of 55mm. For comparison with deformed bar the corresponding results of series AV through to AZ were used along with those for a working bond stress of 2.45 N/mm² (i.e. series C).

4.1.2.5 Load Cycling: - A series, E, was carried out under test condition (c) using deformed bar and 55mm cover. After the heating and cooling periods had been completed the load was cycled 20 times between 3.70 and 1.0 N/mm² and then increased to failure.

4.1.2.6 Concrete Strength: - A test series, F, was undertaken to obtain the variation in the concrete compressive strength with respect to temperature. Concrete cylinders of the size of the 55mm cover bond specimens were used for this purpose. These specimens, four per batch for a particular temperature level, were prepared from the same nominal mix as the pull-out specimens, underwent the same curing period and heating cycle, and were heated under no stress and then loaded to failure after cooling had taken place.

After the completion of a bond test a specimen was tested in indirect tension and subsequently a tensile test was carried out on the steel bar to monitor the influence of temperature on its strength characteristics. Finally a few pull-out tests were performed with acoustic emission transducers attached to the specimens in order to study the relationship between bond slip and emission.
4.1.3 Comments on the Test Programme

4.1.3.1 Standard Specimens

Apart from the exceptions of the varying covers and the plain bar tests all the specimens were of 55mm cover and deformed bar. Therefore a basic or standard specimen was adopted so that comparisons between the various parameters could be made. The deformed bar was chosen as it is more widely used in practice compared with the plain reinforcing bar. The 55mm cover was used since it was easier to cast the specimens compared with those of smaller covers, the larger the dimensions the less delicate the procedure becomes, which in turn gave a greater probability of obtaining consistent results. In addition to this the 55mm cover gave a greater opportunity of reaching the 750°C temperature level while under load during the heating-up period. As the results show it was the only cover which reached this temperature while heated under stress, and therefore its choice as the basic specimen size was vindicated.

4.1.3.2 Test Conditions

Test condition (c) i.e., stressed residual, was used throughout when considering cover, type of bar, bond stress and load cycling. The reason for this is its relevance to conditions found in actual structures. In view of this it might be asked why were test conditions (a), (b) and (d) examined? The thinking behind this decision was as follows. Condition (c) provides information on the bond strength when designing for the reinstatement of fire damaged structures whereas condition (a) is more appropriate to the initial design of structures for a specified fire resistance. Conditions (b) and (d) do not represent any situation found in practice but they do enable the stress and no stress conditions to be compared as has been done for concrete strength at elevated temperatures and also are essential for calculating the slip for conditions (a) and (c) during the
heating cycle. The bond specimen had a section of unbonded steel and concrete between the bond interface and the slip measuring arrangement, therefore, to accurately assess slip the movement over this unbonded section had to be taken into account. This correction factor was made available from the movement measured during the heating under test conditions (b) and (d). It was necessary to consider each temperature level separately due to the differences in the thermal conductivity and thermal expansion of the concrete throughout the temperature range. Another reason for considering condition (d) was for comparisons with the concrete cylinder strength tests. The furnace set up could not apply load to the concrete cylinders during heating hence they were tested for residual concrete strength following the procedure for test condition (d).

4.1.3.3 Bond Stress

For small bond specimens the maximum bond stress available is far greater than for longer embedment lengths due to the non-uniform stress distribution. In view of this it was not possible to relate the design code directly to the test under consideration. However guidance was taken from CP110, Table 21(23) on deciding upon the steady state bond stress to be applied. The maximum recommended value for deformed bars type 2 with a concrete strength of 35.0 N/mm² is 3.70 N/mm². It was decided to utilise this value as being the worst possible situation recommended by the code and also recognising that the actual bond resistance available in the specimens far exceeded this supposedly ultimate value. Therefore although the code of practice was not applicable strictly to these specimens, it was consulted to obtain the right order of bond stress to apply during the tests. A similar argument applies for the 2.45 N/mm² applied to the plain bar specimens, and this value was used also with the deformed bar, test series C, for purposes of comparison.
4.1.3.4 Concrete Strength Specimens

The residual concrete compressive strength at the various temperature levels was obtained using cylinders the size of the 55mm bond specimens. There were two reasons for this choice of procedure. Firstly, as has been noted, the 55mm cover was taken to be the basic specimen size. Secondly, the compressive strength available from a concrete sample varies with its size and shape (e.g., cylinders give lower values than cubes), therefore to obtain an accurate assessment of the comparative strength of the bond specimens it was necessary to use test samples as near to their size and shape as possible.

4.2 MATERIAL CONTROL TESTS

Before dealing with the bond results themselves the compressive and indirect tensile strength of the concrete and the steel tensile properties were considered.

4.2.1 Concrete Strength Results

4.2.1.1 Concrete Compressive Strength

After the one hour dwell at the maximum temperature a thermal gradient remained throughout the section of the circular cylindrical control specimens (see Figs. 27 and 28). To allow for this the mean temperature throughout the specimen was estimated, hence the results given in Fig. 29 for test series F are the strength values plotted against the mean concrete temperature.

The general shape of the curve is much as would be expected with a rapid drop in strength after about 300-350°C. This is consistent with the changes taking place within the concrete due to loss of moisture and thermal movements over the temperature range (cf. 4.3.2.1). The one unexpected part of the curve is the increase in strength between the 150
and 300°C nominal temperatures. This was most probably due to characteristics of the aggregate used for, as Abrams (9) shows, the aggregate can cause great variations in the strength at elevated temperatures. As will be shown in the bond results this increase was a characteristic that was consistently apparent throughout the test programme. Work by Sager and Rostasy (47) showed this same characteristic to be present for bond tests using quartz gravel aggregate while not apparent for limestone aggregate. This indicates again that it was dependent on the type of aggregate being used.

4.2.1.2 Concrete Tensile Strength

The indirect tensile test was carried out on the concrete cylinder after the bond test had been completed. This served a twofold purpose. It enabled the bond interface to be examined for the mechanism of bond breakdown. Also it gave an insight into the indirect tensile strength of the specimens. This turned out to be valuable information in interpreting the bond results for the different covers.

Two curves are given in Fig. 30. One for plain and one for deformed bars specimens both with covers of 55mm. The plain bar gave the more accurate assessment of the tensile strength, as the effect of the previous bond test on the concrete cylinders was negligible. This contrasted with the deformed bars where, due to the splitting forces, cracking due to the bond tests was apparent especially at the higher temperatures. The effect of this is reflected in the lower tensile strength values for deformed bar specimens.

One of the most surprising results of the project is the difference between these curves and the concrete compressive results of Fig. 29. The shape of curve for the tensile strength is also very consistent throughout the work, for both plain and deformed bar specimens. Therefore it can be stated that for the particular concrete used in this test programme the heating cycle affected the compressive and
tensile strength properties in different ways over the 100 to 250°C range. The fact that the tensile strength, on the whole, deteriorated to a greater extent than the compressive strength agrees with Fig. 31(53) where the expected range of the respective strengths is given for elevated temperatures as opposed to the residual conditions of Figs. 29 and 30.

4.2.2 Steel Tensile Strength Results

A tension test was carried out on the steel bar after heating and bond testing had been completed. Figs. 32(a) and (b) give the results for the plain and deformed bars respectively. The load was applied by an Avery testing machine while the strain was measured with Hottinger Baldwin Messtechnik strain gauge based extensometers type DD1 on a gauge length of 100mm in conjunction with an HBM 225Hz. carrier frequency amplifier and digital indicator types KWS 3050 and DA 3417 respectively. The heat treatment did not affect the residual properties of the plain bar which are shown in Fig. 32(a). The characteristics of the cold worked Tor bar, however, did change due to the heating cycle as can be seen in Fig. 32(b).

When steel is cold worked its yield stress is deliberately exceeded so that large groups of dislocations occur on many slip planes. These dislocations become entangled with each other and are unable to disentangle themselves when the load is removed. This treatment raises the yield stress as the sessile dislocations act as obstacles to dislocation movement on reloading. Therefore a higher applied stress is needed to initiate slip within the crystal lattice compared both with its original state and with mild steel.

When heat is applied a tempering process takes place within the steel up to about 700°C. This produces changes in the crystal structure which tend to relieve locked in strains without the occurrence of full recrystallisation. Above 700°C a change begins to take place in the nature of the crystals.
The ferrite (α iron) and carbon form a solid solution called austenite (γ iron) a process which is reversed on cooling. The change from α to γ iron and back again allows the cold working to be fully removed. On cooling re-crystallisation takes place producing new arrays of strain free crystals.

The temperature at which this transformation is completed varies according to the carbon content of the steel e.g., for a carbon content of 0.25% complete transformation does not take place until approximately 900°C. Therefore if the temperature exceeds 700°C but fails to reach 900°C only partial transformation occurs, both ferrite and austenite being present. This results in only a partial stress relief on cooling. This is the situation for the 750°C nominal furnace temperature in the test programme. The process is called either annealing (partial annealing if complete transformation does not occur) if the steel is cooled very slowly in the furnace or normalising if the steel is air cooled. The heat treatment process in this work would appear to lie somewhere between annealing and normalising.

These processes of annealing/normalising and tempering, account for the change in the stress-strain curve of the deformed bar after the heating cycle. As the temperature increased so the steel gradually reverted to its initial pre-cold work condition.

4.3 BOND RESULTS

4.3.1 Temperature at the Bond Interface

The temperature curves for the furnace and the bond interface, throughout the heating-up period, are given in Fig. 33. These curves refer to the 55mm cover only. There was a drop in the rate of temperature rise at the interface around 100°C. This was due to the evaporation of the free water within the concrete. There was another drop at approximately 530°C caused probably by the dehydration of the Ca(OH)₂ component in the cement paste.
In the results that follow it is the temperature at the bond interface which is plotted. Table 4 gives a complete list of the interface temperatures corresponding to the nominal furnace temperature for each cover. As would be expected the smaller the cover the nearer the bond temperature approached the furnace temperature, and for the 25mm cover at 300 and 450°C nominal temperatures the interface temperature did attain the same level as in the furnace.

4.3.2 Basic Specimen

The results for the standard specimen of 55mm cover, deformed bar and a bond stress of 3.70 N/mm² will be considered first. The bond test was made up of three distinct phases, the heating-up period, the 24 hour cooling period and lastly the loading to failure. Hence the results also were split into three parts, namely, the apparent slip-temperature, apparent slip-time, and bond stress-slip relationships respectively. In addition to these the maximum bond stress-temperature relationship has been established.

4.3.2.1 Apparent Slip-Temperature Relationship

The heating-up period gave a relationship between the apparent slip and temperature. To obtain the apparent slip a correction was required for the movement of the length of unbonded steel and concrete between the transducer and the bond interface. This was achieved by subtracting the movement recorded on specimens with no applied load during heating, from the movement obtained from specimens with loading applied. Figs. 34 and 35 illustrate this point.

The shape of the curve for unloaded specimens depended on the rate of expansion of the steel and concrete with increasing temperature. It is made up of four phases which are explained as follows:-
(a) When the heating commenced it was the concrete that was warmed through first, add this to the fact that to begin with the heating produced an expansion in the cement paste (see Fig. 1), and hence the concrete as a whole, then the concrete expanded at a greater rate than the steel showing up on the curve as an initial positive movement.

(b) The curve moves in a negative direction as the steel began to expand at a greater rate than the concrete. The expansion of the concrete was hindered by the dehydration of the tobermorite gel at temperatures around 200°C and later on by the dehydration of the calcium hydrate which commenced at about 400°C, both of these reactions caused the cement paste to shrink. Although the bond interface temperature was between 100-370°C, during this phase, due to the temperature gradient throughout the specimen, the corresponding maximum concrete temperature was in the range 150 to 450°C.

(c) At about 375°C on the curve another positive movement is seen. This is due to the rapid increase in the concrete expansion caused by the transformation of the $\alpha$-quartz to $\gamma$-quartz at temperatures of around 450-575°C(5). The rate of the increase became greater when the whole of the concrete section entered the temperature range of the most rapid period of expansion i.e., 500-575°C. This steep section is seen to end at a bond temperature of 575°C. However the concrete continued to expand faster than the steel up to about 600°C as the quartz transformation continued to take place over a constantly decreasing central core of the specimen.

(d) At 600°C the transformation being complete the steel once again expanded at a greater rate than the concrete showing up as the final negative movement.

The dashed lines on the graphs represent the movement after the required furnace temperature was reached and as it was held constant for the one hour dwell period.

From the movement in the loaded and unloaded specimens
the apparent slip-temperature relationship as seen in Fig. 35 was obtained. The apparent slip increased up to 100°C due to the disturbance of the bond caused by the difference in the expansion coefficients of the steel and concrete as mentioned above. After 100°C the rate of apparent slip decreased slightly which could correspond to the smaller gradient, at these temperatures, of the unloaded curve in Fig. 34 as compared with the initial gradient of the same curve. In other words the differential movement due to expansion between the steel and concrete was reduced at this point which was reflected in a smaller disturbance to the bond and hence a reduction in the rate of apparent slip. However the apparent slip did continue to increase steadily up to about 450°C at which point it began to increase rapidly probably due to both the dehydration of the calcium hydrate and especially the rapid expansion of the aggregate.

It could be that these reactions cause the concrete strength to reduce to such an extent that crushing failure of the mortar immediately beneath the ribs, due to the force from the applied bond stress, took place at this point. This would help to account for the tremendous increase in the apparent slip that followed and is an argument that is considered in more detail in section 4.3.2.3.1. Evidence that the expansion of the aggregate aided the breakdown of bond is further seen in the way the increase in slip was reduced after a bond temperature of around 600°C was reached.

In Fig. 35 it is the apparent slip that has been plotted against temperature as the slip value was a combination of true slip and creep movement. Therefore it is clear that the increase in apparent slip with temperature was also partly due to the corresponding rapid increase in creep, at elevated temperature levels (cf. Fig. 6). To obtain the true slip value an estimate of the breakdown of the two contributory components had to be made. It was not possible to do this experimentally. However an estimate can be deduced theoretically by comparing the bond stress-slip results from the tests carried out with
and without a steady bond stress during heating. This is dealt with in 7.2.1.1. after the bond stress slip results have been discussed.

Another point which arises from the curve is the way the dashed lines for the one hour dwell at the lower temperatures, although giving slightly lower slip values, follow very closely the complete curve for the 750°C nominal temperature. In view of this it can be considered that the maximum temperature curve gives an adequate representation of the apparent slip-temperature relationship, eliminating the need to test unloaded specimens for each individual temperature of each series. The maximum temperature value being sufficient.

The rate of temperature rise of the bond interface during the period of constant dwell was reduced compared with the period when the furnace temperature continued to increase at 2°C/min as Fig. 35 makes clear. Therefore an important conclusion can be drawn, that the smaller rate of temperature rise produced a slightly smaller slip value for a given temperature which indicates that the rate of temperature rise could be a significant variable over a large range. However for the small differences encountered in this investigation it was not significant enough to make any impact on the results.

4.3.2.2 Apparent Slip-Time Relationship

After the heating-up period was completed the 24 hour cooling down phase began. In this phase the relationship between apparent slip and time has been plotted.

The movement for the loaded and unloaded specimens has been plotted in Fig. 36. As in 4.3.2.1 above the curve for the unloaded specimens depended on the relative movement of the steel and concrete throughout the 24 hour period. When the furnace was switched off and cooling began there was an initial rapid fall in temperature causing the aggregate and hence the concrete to contract. Meanwhile the steel being
surrounded by concrete was protected from such a sudden
temperature change with the net effect being the initial
negative movement shown in Fig. 36(a) for all temperature
levels. This negative movement was greater the higher the
temperature up to 600°C as the higher the temperature the
greater the expansion and hence the greater the contraction
on cooling. After the initial rapid drop in temperature the
rate of decrease slowed down and the steel bar contracted at
a faster rate than the concrete causing the positive movement
seen in Fig. 36(a). Most of the curves end up by levelling
out showing only a very slight increase in movement after
about 10 to 12 hours of cooling. The exception is the one
for the 750°C nominal temperature level illustrating that
movement continued over the complete 24 hour period.

After the 24 hours the concrete and steel contraction
more or less balanced out for the 150, 300 and 450°C curves.
However with the 600°C curve the concrete contracted more
than the steel due to the influential quartz transformation
that had taken place causing considerable contraction to
occur on cooling. The 750°C curve does not follow the
pattern of the others probably because so much cracking had
taken place within the concrete, due to the relative movement
of the cement paste (shrinkage) and aggregate (expansion),
that on cooling it proved to be irreversible(7). This would
result in greater contraction of the steel than the concrete
leading to an overall positive movement.

It is also of interest to establish the overall movement
of the unloaded specimens from the beginning of the heating
up period to the end of the 24 hours of cooling. To do this
the curve in Fig. 34 must be compared with those in Fig. 36(a).
It can be seen that for the 300 and 450°C furnace temperature
levels the net movement was negative while for the 600 and
750°C temperatures there was a net positive movement. This
trend follows an expected pattern. If the steel resumed its
original length the net movement value depends on the concrete,
which for lower temperature levels had an overall contraction
caused by the aggregate returning to its original size while the cement paste did not recover from the shrinkage undergone during heating. At the higher temperatures there was an overall expansion caused by the irreversible cracking mentioned already.

It can be noted that this irreversible cracking was much more pronounced for the 750°C temperature compared with the 600°C level. There are two possible reasons for this behaviour. Firstly the 750°C level being higher would cause more internal cracking to take place in the concrete. Secondly the rate of temperature rise was maintained for a longer time in order to attain the 750°C causing more movement, as seen in Fig. 34, and giving a greater opportunity for cracking to take place.

The 150°C curve gives an overall positive movement due to the initial expansion of the cement paste (cf. Fig. 1) as opposed to its overall contraction at the higher temperatures.

From the movement curves, Fig. 36, the apparent slip was obtained - Fig. 37 gives the apparent slip-time relationship for the cooling period. As would be expected the greatest apparent slip occurred during the early part of the cooling period when the bond interface was both at its highest temperature and being subjected to its greatest rate of change of temperature. As time progressed and both the temperature and its rate of change decreased so the apparent slip decreased and levelled off for all except the 750°C curve, which is so weakened that the slip was continuous throughout.

4.3.2.3 Bond Stress-Slip Relationship

After cooling the bond specimens were loaded to failure giving the bond stress-slip relationships shown in Fig. 38. This family of curves was obtained directly from the load
applied and the slip measured. The slip was small to begin with but gradually increased with the bar being pulled through under the maximum bond stress. Except for the $300^\circ C$ curve the residual bond slip increased with maximum temperature level for a given bond stress. This was consistent with the corresponding decline in the properties of the concrete (see Fig. 29).

The shape of the bond stress-slip relationship shows two distinct stages which were particularly clear at the lower temperature levels of 20-300°C. First there was an initial increase in stress to approximately 5.25-5.75 N/mm² for only a very small slip of the order of 0.0013-0.008 mm which can be seen in the localised enlargement of Fig. 38 given in Fig. 39. At this point the curve reaches what can be called a critical value. The second stage began with the stress continuing to increase accompanied by a significant jump in the rate of slip. As failure approached the stress began to level off with the slip still increasing. These two stages can be explained as follows:

**Stage 1**

Fig. 40 shows the various stresses at the bond interface. During stage 1 as the stress increased the slip was very small because in this period no failure in the concrete had taken place. It was not until the critical value was reached that the normal stress under the ribs exceeded the concrete crushing strength causing failure to occur and thus initiating a significant increase in the slip value. It was, therefore, the concrete in compression immediately beneath the ribs that initially failed. To help verify this the stress under the ribs was calculated for different temperatures at the critical value using the formula.

\[
\text{normal stress in concrete under rib} = \frac{\text{force from rib}}{\text{rib area projected on a normal plane}}
\]

where

\[
\text{force from ribs} = \frac{\text{total force}}{\text{applied}} - \frac{\text{force due to sliding resistance of bar surface}}{
\text{}}
\]
The complete calculation for ambient temperature is given in appendix 2. The actual sliding resistance of the smooth portion of the deformed bar is not known, however, an estimate was made by utilising the plain bar results. In addition to this, as only one test condition was examined for the plain bars an adjustment had to be made to these values to make them compatible with the deformed bar results which were tested under a number of different test conditions.

For the basic deformed bar specimen now under consideration the adjustment was for different steady state bond stresses (e.g., deformed, 3.70 N/mm²; plain, 2.45 N/mm²). To apply this adjustment it was assumed that at the critical value of slip the variation in the bond stress for plain bars with a test condition was directly proportional to the variation in the bond stress for deformed bars under the same test conditions. The datum values, from which this variation was measured, were the deformed and plain bar results obtained under the test condition of a steady state bond stress of 2.45 N/mm² during heating and loaded to failure when cooled. This assumption is set down in more detail in appendix 3.

The normal compressive stress under the ribs was expressed in terms of the concrete cube strength at ambient temperature and is shown, for different temperatures in Fig. 41. The curve for the strength under the ribs follows the same pattern as that for the experimentally obtained concrete strength curve, indicating that the critical value was dependant on the concrete strength. As might be expected the computed strength beneath the ribs was greater than the corresponding cube strength due to the small area and restrained conditions of the former case.

It must be noted, however, that the two curves are not directly compatible. The $\sigma_{cu}$ values are for unstressed specimens while the $\sigma_{cu}$ values are for stressed ones. From existing work done (8,9) on concrete at high temperatures, if the stressed condition had been used, the $\sigma_{cu}$ curve would
be increased slightly, but not by a sufficient amount to alter the deductions above. This was confirmed by a comparison of the $\sigma_{cn}$ values for unstressed specimens and the corresponding concrete strength shown in Fig. 58. Again the $\sigma_{cn}$ values were generally speaking considerably greater than their $\sigma_{cu}$ counterparts.

It was not possible to obtain points on the $\sigma_{cn}$ curve for nominal temperatures of 600 and $750^\circ C$. The reason being that the critical value had already been exceeded during the heating up period before the loading to failure took place. This is shown in appendix 4 where the computed stress under the ribs for the steady state bond stress at $600^\circ C$ is far in excess of any $\sigma_{cn}$ value expected. The apparent slip-time curve is consistent with this argument. As seen in Fig. 35 the curve gives a dramatic increase in slip at around $475^\circ C$, which, it is suggested was the moment when the strength of the concrete under the ribs had been so reduced by the heat that crushing had taken place. Hence this point on the curve is considered to be equivalent to the critical value on the stress-slip curves for the lower temperatures (i.e., $20-450^\circ C$).

From this experimental work it was not possible to arrive at a completely satisfactory assessment of the $\sigma_{cn}$ values. To do this an accurate measure of the sliding resistance due to the smooth portion of the bar is needed. Here this value was estimated from the plain bar tests which could have slightly different surface pitting characteristics which in turn would effect the final result. Also there may have been inconsistencies caused by any inadequacy in the assumption used to adjust the plain bar results. However, although the specific values could not be obtained with total accuracy the overall shape or trend of the results is clearly evident as seen in Fig. 41 and also Figs. 58 and 63 which are considered in more detail later. Therefore it was concluded that the critical point in the bond stress-slip curve could be due to the maximum strength of the mortar immediately.
beneath the rib being attained and that this maximum strength was in excess of the strength obtained from test cubes.

4.3.2.3.2 Stage 2:— The second stage began with the critical value already mentioned. The increase in the rate of slip was due probably to the continued failure of the mortar beneath the ribs. A contributory factor in this behaviour would be the porous nature of concrete which would permit the failed mortar particles to be compressed into the pores.

Rehm (33) put forward the suggestion that for shear stresses in the concrete tooth between ribs in the range 0.4 to 0.6 times the cube strength, a fracture occurs in the direction of the maximum shear stresses. This fracture occurred at slips of the order of 0.055 to 0.1 mm at ambient temperature, causing subsequent large slips to take place for rib spacings of about seven times the rib height.

It seems probable that the critical value marked the beginning of the development of this fracture surface (Fig. 40). If this was so, the increase in slip for the first part of stage 2 would coincide with the continued development of the fracture surface. For a few specimens a further 'kink' in the stress-slip graph was noted. This occurred for values of slip which would be expected on completion of the fracture surface. An example of this is seen in Fig. 90. This graph deals with the acoustic emission results but also shows the 'kink' or step in the curve which, it is assumed was caused by the completion of the fracture surface. The calculation shown in appendix 5 and the subsequent results plotted in Fig. 42 indicate the fracture surface to have been completed by the time the shear stresses reached a magnitude of 0.15 to 0.425 times the cube strength. This is appreciably lower than the range of 0.4-0.6 reported by Rehm. It is likely that this difference was due to the greater rib spacing/rib height ratio (c/a) used here i.e., in this work the c/a value was about 15 for transverse ribs and 76 for helical ribs whereas it ranged from 2 to 10 in Rehm's work. This means that the calculated
shear stresses in the concrete were averaged over the complete tooth length whereas in reality the stress at the top of the tooth would be greater than at the bottom. For smaller c/a ratios this averaging effect would not be so pronounced.

It is interesting to note that the ratio of the maximum permissible shear stress in a beam to the crushing strength of the concrete is, according to Table 6 in CP 110: Part 1: 1972 (23), for a comparable grade of concrete (grade 35) of the order of 0.13. This is below the lower limit shown in Fig. 42 and for ambient conditions demonstrates that there is a safety factor of about 1.5 in the code values.

As Fig. 38 demonstrates, on the whole the presence of this fracture surface did not cause any sudden impact on the bond stress-slip curve, as it occurs according to Rehm (33), only over a range of 5-7 times the rib height. For the relatively large rib spacing used in this work it affects only the upper portion of the tooth, the lower portion prevented excessive slip from occurring immediately.

4.3.2.3.3 Approach of final failure: As the final failure of the specimen approached the increase in the slip could be attributed to a number of contributory factors. Even though it did not show up as a sudden change in the graph the fracture surface must have contributed to the gradual increase in the rate of slip. Also as the stress increased any deformation of the lower part of the concrete tooth would be added to the compression of the mortar under the ribs which was still occurring. On top of this torque from the spiral shape of the ribs could have set up a twisting action which again would contribute to the slip. This twisting action would only be relevant to the final stages of slip when more freedom of movement was available, the different angles to the bar axis of the two rib types preventing any twisting from occurring nearer the start of the test. Finally the fractured concrete became a wedge causing increased pressure. This increased the lateral strain in the concrete.
specimen as a whole especially close to final failure and would again aid slip. Failure of the specimen could be defined as occurring when the transverse pressure caused the surrounding concrete to split. It must be noted however that for the 55mm cover specimens the splitting was not a dramatic bursting apart of the specimen but was instead characterised by at the most hair line cracking at the cylinder surface. At times not even the minute cracks were visible, indicating that the cracks having started at the bond interface ended within the specimen itself.

4.3.2.3.4 Summary:- Having considered the bond stress-slip relationship it is evident that the major cause of slip was the compression of the mortar beneath the ribs. As this continued the concrete tooth began to shear off. This commenced with the fracture but also continued to occur afterwards over the lower part of the tooth, although complete shearing-off of the tooth did not take place. The final failure occurred when the transverse pressures caused splitting to take place within the concrete cylinder.

It has been mentioned that this two stage pattern was not relevant at 600 and 750°C, as the critical value was exceeded on heating up. This accounts for the smooth appearance of the curves at these temperatures.

4.3.2.3.5 Maximum bond stress-temperature relationship:- A notable feature of Fig. 38 is the 300°C curve. Although it begins by showing a greater rate of slip for a corresponding stress compared with the 150°C curve this soon changes and it crosses the 150°C curve giving a maximum bond stress equivalent to that at ambient temperatures. This is also seen clearly in Fig. 43 where the maximum bond stress is plotted against interface temperature. The reason for this could be appreciated by referring to the residual compressive strength and temperature results in Fig. 29. where the same trend is seen. The conclusion that can be drawn is that the
concrete strength was a vital factor and was in direct relation to the performance of the bond strength. Also Fig. 43 shows that the bond stress at the critical value followed the same pattern as for the maximum value for temperatures up to 450°C. One last point to note is that at temperatures above 400°C the bond capacity rapidly decreased.
CHAPTER 5

EXPERIMENTAL INVESTIGATION

COMPARISON OF VARIABLES

5.1 COMPARING VARIABLES.

Having looked at the results for the standard specimen the effect of the different parameters will now be considered. These results are presented in the same three stage format as those above with the slip-temperature curves relating to the heating up period, the slip-time curves to the 24 hour cooling down stage and the bond stress-slip curves to the loading to failure. The maximum bond stress-temperature curves are also plotted.

5.1.1 Apparent Slip-Temperature Relationship

The results for the different test conditions during the heating up period have already been considered in obtaining the apparent slip-temperature curve for the basic specimen, and so further comment here is not necessary. Also the condition of the standard specimen and those tested for load cycling are identical over the heating up period and hence the load cycling variable need not be considered at this point. Consequently three variables remain to be examined, namely cover, steady state bond stress and type of bar.

5.1.1.1 Cover

The effect of cover on the apparent slip-temperature relationship is given in Fig. 44. From the graph a number of points can be noted. If an allowance is made for a slight scatter in the results, the slip for all covers up to about 300°C can be considered to have been the same. However for temperatures above this the slip increased as the cover was
reduced for a given temperature. This relationship was more pronounced for smaller rather than larger covers, and there would seem to be a value of cover beyond which, the apparent slip-temperature curve was unaffected by the amount of the surrounding concrete. This was due probably to the difference in the mode of failure of the small and larger specimens. For small covers there was not as much resistance available to prevent failure by splitting of the concrete. However as the cover was increased the resistance to splitting was greater and hence failure occurred at a higher temperature (e.g., compare the curves for 25mm and 32mm covers). As the cover continued to increase a point was reached where the concrete resistance was such that the failure of the specimen was more by the bar being pulled through rather than by complete failure of the surrounding concrete. Hence a point was reached where the apparent slip-temperature relationship was no longer dependent on the amount of cover but on the breakdown of the concrete at the bond interface. This point was approached with the 46mm and 55mm covers in Fig. 44.

5.1.1.2 Steady State Bond Stress

Fig. 45 shows the effect of bond stresses of 2.45 and 3.70 N/mm² on the apparent slip-temperature relationship. The results for both stress levels follow the same basic shape with the curve for the 2.45 N/mm² stress being at a lower level of slip than the 3.70 N/mm² stress. Another point to come from this figure is that the additional stress of the 3.70 N/mm² level affected the slip to a greater extent as the temperature increased i.e., the ratio of the slip at the 2.45 N/mm² stress level decreases as temperature increases. At 100°C it is 0.77, at 300°C it is 0.60, at 500°C it drops to 0.55 and finally at 700°C it is 0.49. The curve for a steady state bond stress of zero is a line along the x-axis of Fig. 45 as the movement measured was assumed to be due to the difference in the thermal movement of the unbonded length of steel and concrete.
5.1.1.3 Type of Bar.

Fig. 46 gives a comparison between plain and deformed bars. Two comparisons can be made. Firstly between the curves for the maximum bond stress recommended in CP110(25) i.e., plain bar - 2.45 N/mm² and deformed bar - 3.70 N/mm², and secondly the difference in performance when both plain and deformed bar specimens had a bond stress of 2.45 N/mm². As expected the slip for the plain bar increased at a greater rate than that of the deformed bar. The rapid increase in slip started at about 250-300°C and the bar pulled completely through at around 435°C. To highlight the great difference in the two bar types a slip of 0.15mm occurred in the plain bar specimen (steady state stress = 2.45 N/mm² heating cycle) at 175°C while the same slip in the deformed bar specimen with a similar steady state stress level took place at 500°C. Despite this appreciable difference the curves are of the same basic shape, the initial rate of slip decreasing around 100°C-120°C and then increasing rapidly on reaching the critical value, at which the concrete crushes under the ribs.

5.1.2 Apparent Slip-Time Relationship

The comments at the beginning of section 5.1.1 also apply here and so only the variable of cover, steady state bond stress and type of bar will be considered.

5.1.2.1 Cover

The no load movement values were not obtained for the different temperatures at the lower covers and so it was not possible to calculate the apparent slip as was done in 4.3.2.1. For this reason Fig. 47 shows the relative movement between bar and concrete for the loaded specimens plotted against time. However from the movement-time curves the general effect of the cover during the 24 hour cooling period can be seen. A discrepancy is apparent in the 150 and 300°C curves for the 32mm cover in Fig. 47(c) (the actual
temperature at the interface being 120 and 285°C cf. Table 4). For all the other covers the movement increases with temperature but in this case the situation is reversed. If this is assumed to be either an experimental error or an exceptional result the 150 and 300°C movement curves can be considered to be fairly constant for all covers, the slight increase that occurs for the smallest specimen size could be due to the increase in the actual bond temperature. This is in agreement with the result from Fig. 44 that the apparent slip-temperature curve is the same for all covers up to about 300°C. At the higher temperature levels, as the cover decreased, the slip increased over the cooling period. For the 600 and 750°C temperatures failure eventually occurred during heating up (see Fig. 44) while the 450°C curve for a 25mm cover shows that the specimen failed during the cooling down period (see Fig. 47(d)).

5.1.2.2 Steady State Bond Stress

In the case of the specimens with 55mm cover it was possible to calculate the apparent slip and hence its variation with time. For a bond stress of 2.45 N/mm² using deformed bar the results are shown in Fig. 48. The results for the 3.70 N/mm² stress level have been given already in Fig. 37. By comparing these two figures it can be seen that the slip for the 2.45 N/mm² stress was greatly reduced for the 600 and 750°C temperatures, whereas for the 300 and 450°C the difference was small. Once again this demonstrates that the effect of the larger stress, as compared with the smaller one, was more pronounced as the temperature increased.

5.1.2.3 Type of Bar

Fig. 49 gives the results for the plain bar which can be compared with both Fig. 37 and 48. The slip for the plain bar was, as expected, greater than that for the deformed bar. However the most significant difference was the continuous nature of the plain bar curves throughout the complete 24 hour
cooling period for all the test temperatures employed. Although this may level off as time goes on it shows the unsuitability of plain bars in the elevated temperature situation except perhaps for considerably reduced bond stresses.

5.1.3 Bond Stress-Slip Relationship

5.1.3.1 Test Condition

The different bond stress-slip curves are given in Figs. 38, 50, 51 and 52. Figs. 50-52 clearly show the two stages discussed in 4.3.2.3 for temperatures up to 400°C while at higher temperatures the curves become noticeably smoother. It is evident that the most significant drop in the bond performance occurred between 400 and 565°C coinciding with the calcium hydrate dehydration and the quartz transformation. One surprising feature of Fig. 51 is that the bond resistance at 450°C was considerably better than that at 150°C, a point which is commented on below.

From these figures differences in the bond stress-slip relationship at 750°C can be seen but it is difficult to distinguish any marked differences for the lower temperatures, therefore the maximum bond stress-temperature curves for each condition have been compared in Figs. 53 to 57. From these figures two comparisons can be made which are the difference between the stressed and unstressed conditions and the effect of testing the specimens whilst hot and when cooled.

5.1.3.1.1 Stressed and unstressed during heating cycle:- The results have been plotted for the hot and residual specimens separately on Figs. 53 and 54. These figures show only a small difference between the stressed and unstressed conditions. They all follow the basic pattern of the concrete compressive strength-temperature relationship shown in Fig. 29. At ambient temperatures the unstressed value was the greater of the two. It is assumed that this was due to some
disturbance of the bond interface in the stressed specimens caused by the constant steady state bond stress over the 24 hour period before loading to failure took place. Fig. 53 indicates that the unstressed specimens gave a slightly lower bond strength except for the 400°C temperature. Fig. 54 on the whole shows also that the unstressed values were the smaller of the two conditions, the main exception to this being at 250°C. By considering Fig. 55 where the results of all four test conditions are compared it seems reasonable to assume that the two results mentioned above were exceptional, this is especially noticeably for the 400°C temperature level. In view of this it can be stated that at elevated temperatures the stressed gave a slightly better bond performance than the unstressed condition. This follows the accepted behaviour of concrete when exposed to high temperatures (8,9). It is considered to be due to the restraint preventing cracking within the concrete forming to the extent that it would if unstressed, hence reducing the loss in strength suffered.

5.1.3.1.2 Hot and residual conditions:— The same procedure as above was carried out here. The hot and residual results are plotted for the stressed and unstressed conditions, see Figs. 56 and 57. Both figures give the same pattern. The residual specimens showed a greater bond strength for temperatures up to 250°C after which the situation was reversed to the end of the temperature range, where the difference between the two conditions became much more pronounced. It is commonly accepted that the residual strength of concrete is less than that while still hot (8,9). This is explained by the continual movement that takes place during cooling causing the cracking to develop to a greater extent. Therefore the bond performance being dependent on the concrete strength would be expected to follow the same pattern, a point alluded to, but unconfirmed experimentally, by Hertz (43).

One possible explanation for the different nature of the results obtained in this work is that when the specimens were hot, thermal stresses were produced between the steel and
concrete, especially between the ribs of the bar, which were added to the stresses caused by the applied load. Therefore failure occurred at a lower applied loading than for the residual condition where these thermal stresses were not present. This occurred for temperatures up to 250°C at which point the internal changes in the concrete due to cooling would begin to have a greater effect than the thermal stresses when hot, causing the residual bond value to be less than that at the elevated temperature. By the time 750°C was reached, the contraction on cooling would be so great that the difference in the bond strength between the heated and cooled states could increase considerably.

Milovanov and Pryadko(36) reported the bond strength to be greater when cooled rather than hot for temperatures up to 450°C although their work was for refractory concrete with a water glass base.

5.1.3.1.3 $\sigma_{cn}$-temperature: Fig. 58 gives a comparison between $\sigma_{cn}$ values for test conditions (a) (b) (c) and (d) and the concrete compressive strength. The concrete was heated under no stress and loaded residually and is therefore directly compatible only with test condition (d), with which reasonable agreement is obtained. For test conditions (b) and (d) values of $\sigma_{cn}$ can be obtained for all temperature levels as loading to failure commenced from zero (appendix 4). In computing the 600 and 750°C values the plain bar resistance was assumed to be zero. This could account for the increase in the $\sigma_{cn}$ curve for condition (d) between 450 and 600°C as this zero resistance was taken from stressed plain bar specimens. However it is possible that plain bar specimens heated under no stress could retain a little bond strength which in turn would reduce the $\sigma_{cn}$ value e.g., a nominal bond stress of 0.3 N/mm² would give a value of 68.8%.
As the $\sigma_{cn}$ values for test conditions (a) and (b) are not directly compatible with the $\sigma_{cu}$ curve they are shown as broken lines. Their pattern is similar to that in Fig. 55 where the hot specimens give smaller maximum bond stresses for the lower temperatures and then greater values at the higher temperatures. As can be seen from Fig. 58 these two curves come very close to the concrete strength curve and condition (b) even crosses it at 150°C nominal furnace temperature. This is an unexpected result and requires some comment.

The fact that the curves are not directly compatible with the $\sigma_{cu}$ curve does not help as it is generally held, as noted in 5.1.3.1.2, that the concrete strength is greater for the hot than for the residual cool case, which increases rather than lessens this discrepancy. Another possible explanation is that the assumption used in the calculation (appendix 3) breaks down when adjusting between the hot and residual situations. In the previous section it was noted that when hot the thermal stresses between the steel and concrete, especially between the ribs, could be added to the applied stresses causing a lower failure load than for the residual case. If these thermal stresses were greater in the case of the deformed bars due to the ribs, the relationship between the bond stress changes in the plain and deformed bar for the various test conditions and would not be directly proportional as was assumed originally. The effect of this would be to increase the sliding resistance in relation to the total force applied. This in turn would reduce the value which would help in accounting for the anomaly under consideration. However any effect of this inconsistency in the assumption would also be evident at the 300°C temperature level, in which case it would be expected that the $\sigma_{cn}$ values for conditions (a) and (b) would be considerably below that for (d) (see Fig. 55). This is clearly not the case. Also at the higher temperatures of 450-750°C the pattern of the hot and residual $\sigma_{cn}$ curves follow the expected trend.
In view of these considerations it is concluded that any effect caused by the inadequacy of the assumption used is negligible when compared with the natural experimental scatter in the results.

Therefore the most probable explanation for the low values at 150°C and test condition (b) is that one batch of four specimens does not give as accurate an assessment of the plain bar resistance compared with using specimens from different batches.

5.1.3.2 Cover

Figs. 38-59, 60 and 61 compare the bond stress-slip relationship for the four different covers tested. As the cover decreased both the maximum bond stress and the maximum bond slip also decrease. The tremendous difference in the slip between the smaller and larger covers was due to the different modes of failure as mentioned in 5.1.1.1 above. The lower cover values (i.e., 25 and 32mm) begin by following the curve shape as explained in 4.3.2.3. However the build up of pressure would cause the concrete surrounding the steel bar to split before the bar was pulled through. The reduction in the gradient of the curve immediately before failure was caused probably by the increase in the concrete strain through the section at this time. As Fig. 59 shows, the 46mm cover allowed a much greater slip to take place, showing the bar to be pulled through by about 1mm prior to when failure of the concrete eventually occurred. This trend was continued by the 55mm cover where the bar was pulled through at a slip of the order of 2-4mm.

As the cover was reduced so the bond value at 300°C decreased with respect to the other temperature levels. This can be seen also in Fig. 62, where the maximum bond stress-temperature relationship for the different covers is shown. Once again this was due to the change in the failure mode.
This could be appreciated by comparing Figs. 29, concrete compressive strength-temperature and 30, concrete splitting strength-temperature, with Fig. 62. The 46 and 55mm covers, both of which were associated with pull-through bond failure, follow the shape of the concrete compressive strength curve, hence the maximum bond stress was dependent on the concrete compressive strength at the bond interface. The 25 and 32mm covers, however, follow the concrete splitting strength curve indicating that in this case the maximum bond stress was affected far more by the strength of the surrounding concrete. These two reactions must not be considered to occur totally separate from each other. The lower bond values of the 46mm compared with the 55mm cover might suggest that both of these reactions were involved in the behaviour of the 46mm cover specimen size.

The dashed lines in Fig. 62 represent the fall in the bond stress to the steady state stress level at the next temperature. This value was not reached in the test due to the failure of a specimen before loading to failure could take place.

5.1.3.2.1 $\sigma_{cm}$-temperature: Once again the values follow the general shape of the concrete compressive strength curve (Fig. 63). The unexpected results here were the low $\sigma_{cm}$ values for the 25mm cover at 20 and 150°C. One possible explanation for this could be that the concrete around the interface was not compacted sufficiently well compared with the other covers, due to the operation being more difficult to complete for the smaller covers (cf. 4.1.3.1).

5.1.3.3 Steady State Bond Stress

This has already been considered under the test condition parameter in 5.1.3.1.1 but some additional comments on the 2.45 N/mm² bond stress series are necessary. The results are plotted on Figs. 64 and 65 and are compared with those for bond stresses of 3.70 N/mm² and zero (Figs. 38, 52 and 54).
The 300°C value for the 2.45 N/mm² series is taken to be an exceptional result. This is because the points for the 3.70 N/mm² and zero bond stress curves were the mean of five specimens, each taken from a different batch of four specimens each (cf. 4.1.2.1), whereas the 2.45 N/mm² results were the mean of four specimens all of which were from the same batch. Hence it is possible that the low 300°C value was due to this particular batch of four specimens giving lower results than the others used. This was confirmed by checking the quality control cubes for the batch which also showed exceptionally low strength values.

At the higher nominal temperatures of 600 and 750°C the 2.45 N/mm² bond stress produced a lower maximum bond value than that for the 3.70 N/mm² stress, which was to be expected from the results discussed in section 5.1.3.1.1. However the 2.45 N/mm² values were lower than those for the unstressed specimens, which was contrary to normal expectations. The reason for this might be as above, that the results were obtained from four specimens of the same batch instead of from different batches, which would have kept to a minimum any slight variations in the concrete mix and test environment.

Therefore in terms of understanding the effect of differing steady state bond stresses applied during heating, this series was not a success. However it does show the need for care in comparing results, even though the concrete mix and testing procedures were nominally equivalent, as any errors or slight variations in the test would add to the scatter in the results expected in experimental work of this sort and could cause misleading conclusions to be drawn. This point becomes less significant as the difference in the results becomes greater e.g., it would be valid still to compare the 2.45 N/mm² bond stress deformed bar results with the corresponding values obtained for the plain bar.
5.1.3.3.1 $\sigma_{cn}$-temperature: This set of values for $\sigma_{cn}$ did not require to be adjusted, as both deformed and plain bars were tested under the same set of conditions. As Fig. 66 shows the trend of the two curves is the same. Once again a low value at 150°C is apparent, which confirms that the adjustment was not responsible for this as stated in 5.1.3.1.3. Also out of all the curves this one has the most pronounced saw tooth of any and is far greater than the corresponding one for $\sigma_{cu}$. At first sight this was contrary to what might be expected. As this was the basic datum point for the curves it could be expected to be the most reliable. By comparison the adjusted values for the other test conditions would stand more chance of having a greater scatter. The reason this was not the case stems from the fact mentioned in the section above about the mean of four specimens from one batch. This was the case for both the deformed and plain bars in computing the curve in Fig. 66. By comparison with the other $\sigma_{cn}$ curves in Figs. 41 and 58 it could be appreciated that by using the approach of 4.1.2.1 the mean values for test conditions (a) - (d) would be that much more reliable. It follows that if a similar procedure was adopted for the plain bar tests the $\sigma_{cn}$ curves would again be a little more accurate. Also it must be borne in mind that only a relatively small difference in the bond stress values could cause quite a large change in the $\sigma_{cn}$ curve (cf. example in 5.1.3.1.3).

5.1.3.4 Type of Bar.

The bond stress-slip results in Fig. 67 reflect the lower bond resistance of the plain bar compared with the deformed steel. The bond value depends on the adhesion and frictional resistance available. The frictional resistance in turn depends on the extent of the surface roughness and the tolerance of the lateral dimensions of the bar. As for the deformed bar, the bond stress-slip curve shows a critical value which, in this case, was due probably to the mortar extending into the smaller bar pitting being sheared off.
The increased bond stress beyond this point was due to a combination of the friction caused by the concrete shrinkage, any slight variation in the lateral dimension of the bar and the resistance of the larger, wider mortar peaks. Fig. 68 shows the basic pattern of the maximum bond stress-temperature curve following that for the 55mm cover deformed bar specimen indicating that it was related closely to the concrete compressive strength at the interface. This agrees with the comments above on the effect that the mortar peaks have on the bond performance.

5.1.3.5 Load Cycling

Figs. 69 and 70 show the effect of cycling the bond stress twenty times between 1.0 and 3.70 N/mm² before loading to failure. Results for the 20-450°C temperature levels are given in Fig. 69. The cycling process produced a series of hysteresis loops which agrees with the ambient temperature work of Edwards and Yannopoulus (35). Increasing the temperature results in the gradient of the stress-slip relationship decreasing as shown on previous figures. It also caused the slip at stresses of 1.0 and 3.70 N/mm² to increase at a greater rate for each succeeding cycle. This behaviour occurred between 20 and 150°C but was particularly evident between 300 and 450°C, which appeared to be a very significant stage with considerable irreversible slip taking place at the 450°C level.

The majority of the 450°C specimens gave results comparable to those presented in Fig. 69. However, Fig. 70 shows one of considerably greater slip along with the 600°C results. Although the slip values were much greater the basic shape of the curves is similar to those in Fig. 69.

The reason for the increased slip values could be connected with the critical value that has already been discussed. It has been noted that for the concrete quality adopted and a steady state bond stress of 3.70 N/mm² this critical value was attained at temperatures of approximately
450-475°C. Therefore it could be that the seemingly exceptional result at 450°C shown on Fig. 70 was due to this particular specimen either having a lower concrete strength value or less well compacted concrete in the region of the bond interface, which caused the critical value in this case to be reached at a lower interface temperature of around 400°C. If this is so the sudden increase in slip at about 3.0 N/mm² for the earlier cycles could be explained as being due to the stress in the mortar beneath the ribs having attained or just exceeded its strength at that point. As the number of cycles increased this dramatic increase became much less noticeable. If this argument is correct it could be expected that the same pattern would be apparent and more pronounced for 600°C, which as Fig. 70 shows, is exactly what did take place.

One of the most noticeable trends on Fig. 70 is the way the rate of increase in slip decreases with each additional cycle. If the argument above concerning the critical value being exceeded is correct then this phenomenon can be explained as follows. With each additional cycle the failed mortar beneath the ribs could be compressed into the pores and the depth of failure would be increased slightly. This in turn means that as the cycles continued the failed mortar would be slowly compacted and supported from beneath by sound concrete, which was increasingly unaffected by the cycling as the load was dissipated over a larger depth of concrete.

Fig. 71 shows that the load cycling decreased the maximum bond stress obtained for the specimens. The dashed line to 720°C indicates that the specimens failed under loading at 3.70 N/mm² before the sequence of twenty load cycles and loading to failure was completed. In view of this it would seem that the difference in the maximum bond stress between cycled and uncycled specimens becomes greater for the higher nominal temperatures of 600 and 750°C. This emphasises, as was mentioned in 5.1.1.2, that a change in the test
conditions affected the results to a greater extent at the higher temperature levels.

5.2 CRACK FORMATION OF BOND SPECIMENS

The general pattern of cracking in the tested specimens was consistent over all the test series. However the magnitude of the cracks varied with the size of the concrete cover.

5.2.1 General Crack Pattern

Figs. 72(a) and (b) and the photographs Figs. 73-78 show the general pattern of cracking that occurred. Figs. 72(a) and 73-77 give the most predominant case whereas Figs. 72(b) and 78 indicate a more advanced form in the cracking process which was apparent for some specimens.

Fig. 72(a) shows two longitudinal cracks one either side of a transverse crack. The longitudinal splitting process has been discussed in 4.1.2.1 where it was stated that it was due to the transverse pressure caused by the wedging action of the bar at the bond interface. The hoop or transverse crack was due probably to the distribution of compressive stresses in the region of the embedment length. Fig. 79(a) indicates a compressive stress on the concrete below the rib at the top level of the bond length whereas the mass of concrete above this level and away from the rib resists the tendency of the layer to deform downwards. Hence it is likely that as the load was applied a transverse crack could be initiated at this level which would extend gradually towards the concrete surface as the load was increased. It could be that the most developed portion of this crack would cause the weakness in the concrete cylinder, which predetermined the position of the longitudinal crack regularly situated on either side at the surface.

Fig. 72(b) illustrates the more advanced form of cracking with another transverse crack above the bond interface.
in the portion of concrete not directly loaded. The probable cause of this crack could be understood from Fig. 79(b) in which a mechanism similar to that of Fig. 79(a) is shown. At 600°C the concrete tensile strength would be weakened sufficiently to allow the cracks in the upper portion of a specimen to propagate. Another feature of this advanced form of cracking is that centrally placed transverse cracks could continue to develop across the longitudinal cracks.

5.2.2 Effect of Cover

The four different covers tested can be divided into two groups i.e., the 55 and 46mm in one and the 32 and 25mm in the other. The first pair showed a definite change in the magnitude of the cracking over the temperature range. At ambient temperature small hairline cracks occurred with the occasional exception (see Figs. 74(a) and (b)). Then as the temperature increased so did the extent of the cracking. This continued up to 600°C at which the advanced form of cracking mentioned above became evident. Then for the 750°C nominal temperature level the extent of the cracking due to loading was considerably reduced compared with that at 600°C, although the hairline cracking caused by the heating cycle was much more in evidence (see Fig. 80). This pattern occurred for both the 55 and 46mm covers although the size of the cracks was correspondingly larger for the smaller cover.

This trend demonstrated that as the temperature increased up to 600°C the reduction in the tensile strength caused the restraint, made available by the cover, to be decreased sufficiently to allow greater cracking to take place. This indicates that the decrease in the tensile strength of the concrete was greater than the corresponding reduction in the bond quality, a point alluded to in 5.3.2. The 750°C specimens showed a decrease in cracking. At this temperature it was likely that the heat had caused so much deterioration of the concrete to take place that the breakdown of bond at the interface occurred for small loads which did not cause
such substantial splitting to arise.

For the two smaller covers the increase in cracking with increase in temperature did not occur. In fact the situation was reversed. At higher temperatures very slightly less cracking occurred. This was due probably to the reduced restraint of the small cover, causing considerable splitting to occur at all temperatures but especially at the lower levels of 20 and 150°C. It is possible that this could be accounted for by the change in stiffness of the concrete causing the higher temperature failures to be slightly less brittle than the ones at lower temperatures.

The different emphasis in the failure made for the larger and smaller covers was discussed in 5.1.1.1. From a study of the crack failure pattern it would appear that the different modes were affected in different ways at elevated temperatures. For the large covers the restraint could constrain the cracking at the lower temperatures. Then as the temperature increased the decrease in the tensile strength would reduce this constraint allowing greater cracking to occur. For the smaller covers the restraint available would not be sufficient to prevent large cracks at lower temperatures causing a brittle failure. As the temperature increased the cracking continued to take place but due to the reduction in the stiffness of the concrete a slightly less brittle failure could occur causing the cracks to be smaller at the higher temperatures.

5.2.3 Additional comments on cracking

In chapter 2 the work of Broms and Lutz (25) was noted where the cracks did not always reach the outer surface of the concrete. This was also observed in the present work. Fig. 81 gives a diagrammatic representation of the radial cracks in a specimen starting at the interface but not quite travelling the complete distance to the surface.
Little cracking occurred for the plain bar specimens. There was no cracking due to the bond test but rather hairline cracks (similar to those shown in Fig. 80) were apparent for those subjected to high temperatures caused by the thermal movement within the concrete itself.

Fig. 82(a) shows the deformed bar after being removed from the test specimen, with concrete still adhering to it, while Fig. 82(b) shows the bond length of the specimen exposed after the indirect tensile test was carried out.

5.3 COMPARISON BETWEEN BOND AND CONCRETE STRENGTH

5.3.1 Compressive Strength

It was noted from previous work (appendix 1) that a number of investigators concluded that the reduction in bond strength for elevated temperatures was greater than that for concrete compressive strength. Fig. 83 gives the comparison of the two for the present work. As can be seen these results on the whole confirm this conclusion. The only exception is at the 300°C nominal temperature level and was probably due to experimental scatter. Also it has been stated that the bond is closely related to the concrete compressive strength—a statement which is supported by the shape of the two curves.

The question has to be asked, why was the reduction in bond quality greater than that of the compressive strength? It would seem likely that this was due to the presence of the steel. The concrete strength changes at elevated temperatures due to the interaction between the aggregate and the cement paste, caused by the differing thermal movements induced. When bond is considered the steel has also to be taken into account, hence there are three different thermal movements present which could cause an increase in thermal stresses and subsequently additional cracking and breakdown of the concrete in the vicinity of the concrete/steel interface.
5.3.2 Tensile Strength

Also shown on Fig. 83 is the tensile strength curve obtained. As can be seen the tensile strength decreased at a greater rate than both the bond and compressive strength after the 150°C nominal temperature. It has already been noted that this could be the reason for the change in the extent of cracking occurring throughout the temperature range for the larger covers. It follows that this effect is relevant to the state of cracking in reinforced concrete members after heat treatment, and as such could be worthy of further investigation. It has been stated that a number of investigations including the present work have reported the bond quality deteriorating to a greater extent than the compressive strength. If the tensile strength deteriorates even more, as it would appear from the results obtained, it follows that cover to the reinforcement in, for example, a beam could be a critical area when estimating the fire resistance or the post-fire properties of reinforced concrete members.
6.1 INTRODUCTION

Acoustic emission (A.E.) is a non-destructive method of testing materials and structures. It is based on the fact that materials when under stress emit sounds associated with elastic stress wave release as cracks develop. These sounds can be listened to by small piezo-electric accelerometers, which in turn convert the sound energy into an electrical signal.

The specific application of acoustic emission is in locating defects that are growing. If the defect is not increasing it is unidentifiable by this method. Therefore acoustic emission can be considered to be complementary to ultra sonic testing which can locate cracks but does not give information on their growth. It is as the defect or crack is developing or increasing in size that the bursts of energy are released in the form of stress waves which are subsequently picked up by the transducers.

As with the application of load and the measurement of slip, because of the temperatures involved in this work the A.E. transducers had to be located outside the furnace chamber itself. Therefore a method of transmitting the A.E. signals from the bond specimens to the transducer was necessary. How this was achieved in practice is considered below.

6.2 TEST INSTRUMENTATION

It was possible to monitor the A.E. signal by placing the piezo-electric accelerometer between a fused silica rod and the displacement transducer (see Fig. 84). Water pump grease was used to seal the join between the accelerometer and
rod, while, due to the high temperature requirements Fortafix, a high temperature adhesive cement was adopted to bond the silica rod to the steel bar. Hence the A.E. signals could travel up the steel bar through the fused silica rod and be collected by the transducer.

This proved to be a successful arrangement. When compared with the situation where the transducer was placed directly on the steel bar at ambient conditions 97% of the acoustic emission was found to travel up the fused silica rod to the transducer.

Fig. 85 gives an outline of the instrumentation used. The A.E. signal was split after the pre-amplifier enabling both a RCS Ltd., model 401M frequency counter capable of measuring at a rate of 50MHz., and a datalab, type DL902 transient recorder to be used simultaneously. The frequency counter monitored the number of counts throughout the test while the transient recorder stored 2.048ms of waveform which then could be displayed on an oscilloscope. Photographic records could be obtained at specific points throughout the test.

One drawback to using this frequency counter was that no threshold level could be set, resulting in all the signals triggering the counter being monitored. Later on this setup was improved by the acquisition of an event counter (Endevco model 310). This enabled events to be counted which exceeded a suitable threshold level (in this case 1V after amplification). However the tests completed in this work were initial studies to ensure that the system of transferring the signals from the concrete/steel interface to the transducer was feasible, hence the frequency counter was employed. The results obtained were encouraging leading to the acquisition of the event counter which has been used subsequently by another research student on similar bond pull out specimens but with an 8mm diameter deformed bar. A comparison between the two studies is given in more detail in appendix 6.
6.3 RESULTS

In all fourteen specimens were tested using the A.E. technique. Four main temperatures were considered i.e., 20, 150, 300, and 450°C. At each temperature three specimens were tested one plain bar and two deformed bar samples. They were all heated while under a steady state bond stress and loaded to failure when cool. One of the deformed bar specimens was subjected to twenty load cycles (cf. 4.1.2.5) before increasing to the ultimate stress. In addition to these twelve tests, two extra deformed bar samples were heated to 600°C while under stress and then loaded to failure in the residual condition.

It should be appreciated that this programme was capable of giving only an indication of the A.E. signals present during the bond test. Each condition was tested on an individual specimen, hence it was not possible to obtain mean values or conclusive experimental results. However a significant indication of the relevance of the A.E. signals to bond tests could be obtained as is shown in the comments that follow.

As in 4.3.2 and 5.1 the results are presented in the three stage format corresponding to the heating period, the 24 hour cooling period and the loading to failure.

6.3.1 A.E. Counts-Temperature Relationship

The A.E. counts-temperature graph for the heating period is given in Fig. 86 for deformed bar. The shape of the curve is very distinctive. Up to 175°C there was no significant emission. Between 175 and 325/400°C the A.E. signals began to increase and at around 340/430°C a dramatic increase in the number of counts occurred. Comparing these graphs with the corresponding apparent slip-temperature curve (see Fig. 35) a marked similarity can be seen. The increase in counts at around 175°C coincided with the increase in the rate of slip after a slight levelling off. Also the
rapid increase in counts at approximately $430^\circ C$ (for the 600°C curve) corresponded to the critical value of slip discussed previously (4.3.2.3.1). This increase therefore could be due to the crushing of the mortar immediately beneath the ribs. Hence the A.E. results compliment and add weight to the deductions put forward earlier.

The difference in temperature at which the critical value was reached i.e., $475^\circ C$ in Fig. 35 and $430^\circ C$ in Fig. 86 for the 600°C curve, could be attributed to experimental scatter caused by slight variations in strength, degree of compaction or aggregate particle jamming beneath the ribs. The $450^\circ C$ curve on Fig. 86 suggests that critical slip occurred at an even lower temperature than in the 600°C case and will be considered again in 6.3.3.1.

6.3.2 A.E. Counts-Time Relationship

Fig. 87 gives the relationship between A.E. counts and time for the 600°C test over the first 25 minutes of the cooling period. The rate of occurrence of the A.E. signals was reduced compared with the rate of increase in counts after the attainment of the critical value in the A.E. counts-temperature curve. This could be aided by the reduction in the thermal gradient after the 1 hour dwell at the maximum temperature. After the first 15 minutes of cooling the rate of occurrence of counts began to level off. After 24 hours of cooling the counts were spasmodic. At times emission was clearly in evidence, while at other times there was none at all. This averaged out at a small increase in emission with time. Therefore, just as the A.E. counts-temperature is similar in shape to the apparent slip-temperature curve so the curves for A.E. counts-time and apparent slip-time (cf. Fig. 37) have a similar shape i.e., a rapid initial increase which levels off as time advances.
6.3.3 A.E. Results during Loading to Failure

Figs. 88-92 show the \( \sigma_b \)-slip, \( \sigma_b \)-AER and AER-slip curves for deformed bars at various temperatures. The acoustic emission ratio (AER) is the emission expressed as a proportion of the total accumulated emission counts at the maximum bond stress. All the figures have the same overall pattern. The \( \sigma_b \)-AER curve follows very closely the shape and magnitude of the bond stress-slip relationship, showing that there is a close correlation between the A.E. output and the slip for any given bond stress. The AER-slip curve on the whole has a slightly steeper gradient over the initial stages of slip which then decreases up to about the 2mm slip stage. However the relationship over this range is very nearly linear. For the 300 and 600°C temperatures (Figs. 90 and 92) the linear plot is continued up until failure, whereas for the remaining three temperatures a significant increase in A.E. output is displayed as the maximum bond stress is approached. This is understandable as the strains were increasing, with increasing stress, throughout the concrete as splitting of the cylinder was approached. The 300 and 450°C curves shows that any discontinuities in the bond stress-slip curve are reflected on the \( \sigma_b \)-AER graph and even tend to be more pronounced. It was suggested in 4.3.2.3.2 that this discontinuity (e.g., at about 0.2mm on the 300°C curve Fig. 90) could be where the fracture surface beneath the ribs was completed. On completion the fracture would promote a wedging action which could cause a change in the rate of A.E. to take place i.e., during the development of the fracture surface the rate of A.E. would increase gradually and then reduce as the wedge became restrained by the surrounding concrete. The curves also show that the A.E. could be more sensitive in picking up these stages in the breakdown of the bond, for not only does the A.E. curve show up the discontinuity more clearly than the \( \sigma_b \)-slip curve for 300 and 450°C (Figs. 90 and 91) but on the bond stress-slip curves where the slip does not show up any discontinuity it is possible to discern a change in the \( \sigma_b \)-AER curves in the expected region.
e.g., for the 150°C curve (Fig. 69) there is no trace of any discontinuity on the $\sigma_b$-slip plot but there is a quite discernable change in the A.E. output at a bond stress of approximately 6.0 N/mm². It is also possible to relate the slight change in gradient of the A.E.-slip curve to this same feature.

The $\sigma_b$-slip curve for 600°C (Fig. 92) gives no such kink and is very smooth in shape. This confirms the comments made earlier that the critical value was exceeded and the fracture surface completed during the heating up period, before loading to failure.

Figs. 93-96 give the results for the plain bar specimens, which are very similar to the deformed bar pattern. The number of counts recorded was much less for these tests which is to be expected with no ribs biting into the concrete.

6.3.3.1 Load Cycling Test

Fig. 97 is not a result from any of the fourteen specimens mentioned in 6.3. It represents the behaviour of a trial specimen not necessarily conforming to the basic concrete strength used. It is given here to demonstrate the effect that repeated loading in the cooled condition has on the A.E. On first loading to a given stress level there was a significant number of events produced. However upon reloading the counts value was very small. This was due to the fact that the A.E. technique detects the increase or growth in defects. The signal is only forthcoming if the defect is growing. Hence the small increase on reloading indicated that no further significant damage had taken place. If on reloading considerable emission took place it would demonstrate further damage was occurring due to the reloading process. These points are illustrated also in Fig. 98 where information on the load cycling in the residual condition for tests on the fourteen specimens mentioned above is given.

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The results for temperatures 20-300°C were not plotted mainly because the cycling effect produced no A.E. signals (cf. Fig. 97). The 450°C temperature did however, and it is consequently shown. The curve can be compared with Fig. 70 where the bond stress-slip relationship is plotted. The σ -AER curve naturally shows no hysteresis loops but the similarity between the two is marked, again demonstrating a close relationship between slip and acoustic emission results. This supports the argument that the rapid increase in slip was due to the critical value being exceeded in the load cycling range, with the crushing beneath the ribs causing a considerable increase in the number of counts recorded (5.1.3.5). Also fitting in with this deduction is the 450°C curve of Fig. 86 which shows a rapid increase in A.E. output with a corresponding slight increase in the rate of slip occurring as the critical value was attained during the heating up period (i.e., in this case the critical value was reached before the 450°C furnace temperature was attained). These points would all seem to be consistent with the critical value deduction put forward in 4.3.2.3.

6.3.3.2 Acoustic Emission Waveform

Photographs of the A.E. waveform (see Figs. 99 and 100) were taken by utilising the transient recorder in conjunction with an oscilloscope. They could be taken at specific points throughout a test. The three shown refer to the 450°C temperature. The points at which Figs. 99(a) and (b) were taken for the deformed bar case are shown in Fig. 91. For the deformed bar the maximum peak to peak voltage increased almost four times from 200 to 750mV between points A and B, indicating that cracking was more severe at the end of the test. The photograph for the plain bar specimen (Fig. 100) was taken at the maximum bond stress (see Fig. 96) and shows a reduction of around one quarter compared with the deformed bar at the corresponding point. Both of these trends follow the expected pattern.
6.3.4 Further Comments on Bond Breakdown

Some additional information aiding the understanding of the process of bond breakdown was gained during a visit to the department by representatives of Dunegan/Endevco. Using their own A.E. instrumentation system a few specimens were tested. The tests were conducted at room temperature and two A.E. transducers were attached to the sides of a concrete cylinder on the same generator, one near the top end the other near the bottom. In this way it was possible to locate the position of the A.E. signals within the specimen. Fig. 101 gives a typical result for deformed bar specimens. The great majority of the events occurred over the bond length as expected. The next most active area was the lower part of the cylinder due to the compressive stress applied to this part of the concrete during testing. Very little activity took place in the upper portion of the specimen as this was not under direct loading.

Another interesting point was that at ultimate failure the rate of counts increased first at the loaded end of the bond length and then very rapidly this increase worked its way along the embedment length towards the unloaded end (cf. the end part of the \( \sigma_b \)-AER curve for 20, 150 and 450°C). These tests were carried out on the 55mm cover specimens which it has been noted failed mainly due to the bar being pulled through by about 2-4mm often with cracking showing that a limited amount of longitudinal splitting had taken place (cf. 4.3.2.3.3). The A.E. test with the increase in emission working along the bar from the loaded end also points to this interpretation of the results, showing that the predominant cause of failure was the breakdown in bond at the interface. It would be expected for the smaller cover sizes that this pattern would be less apparent with the emphasis on longitudinal splitting rather than the bar pulling through (cf. 5.1.3.2).

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6.4 CONCLUSIONS AND FUTURE WORK

Although the A.E. tests have been limited they nevertheless give some very interesting results. The A.E. output appears to be closely related to the bond slip. This indicates that the technique could be a useful method of estimating the quality or state of bond in reinforced concrete members. The load cycle results confirm this as the A.E. showed the 450$^\circ$C specimen to deteriorate on load cycling whereas up to 300$^\circ$C this did not occur, which is in line with the conclusions of 5.1.3.5.

In view of this, A.E. techniques could be a very profitable field for future work with the aim of establishing a practical method of estimating the post heat bond properties in fire damaged reinforced concrete structures. Some work on 8mm diameter deformed bar specimens at elevated temperatures has already been undertaken (see appendix 6) and work on small beams with four point loading is in the process of being undertaken as part of another research project by M.R. Khan.
7.1 THEORETICAL APPROACH

7.1.1 Introduction

The bond stress-slip results obtained from the experimental work refer to the local bond properties over the small differential length of 32mm. From this result it is possible to calculate the slip, steel stress and bond stress distributions over a complete length of bar. Rehm(33) followed this approach using his test results at ambient temperatures. In this chapter the procedure is used to investigate the effect of high temperature on the three distributions mentioned.

Having established the effect on the distributions their relevance to the performance of a beam is deduced with particular reference to the end anchorage zone and the crack spacing.

7.1.2 General Differential Relationship

The general differential relationship is given by Rehm(33). Its derivation is set down here for completeness.

The change in displacement $d\Delta$ (i.e. slip) over the distance $dx$ corresponds to the difference between the steel and concrete strains, $\varepsilon_s$ and $\varepsilon_c$, as shown in Fig. 102.

\[
\frac{d\Delta}{dx} = \varepsilon_s - \varepsilon_c
\]

\[
= \frac{\sigma_s}{E_s} - \frac{\sigma_c}{E_c}
\]
where $\sigma_s^c$, $\sigma_c$ and $E_s$, $E_c$ are the steel and concrete stresses and the Young's moduli for those materials respectively.

However, if for the moment the concrete strains are ignored,

$$\frac{d\Delta}{dx} = \frac{\sigma_s}{E_s}$$

Now the change in the steel stress $d\sigma_s$ over distance $dx$ corresponds to the change in force over $dx$ which is related to the bond stress, $\sigma_{bx}$.

$$\frac{d\sigma_{sx}}{dx} = \sigma_{bx} \cdot \frac{u}{A_s} = \sigma_{bx} \cdot \frac{4}{d}$$

where $d$, $A_s$ and $u$ are the bar diameter, cross sectional area and circumference.

To establish a relation between the bar length $x$ and the displacement $\Delta$ a differential equation is derived from (4).

$$\frac{d\Delta}{dx} = \frac{\sigma_{sx}}{E_s}$$

$$\frac{d^2\Delta}{dx^2} = \frac{d\sigma_{sx}}{dx} \frac{1}{E_s}$$

By substituting from (5)

$$\frac{d^2\Delta}{dx^2} = \sigma_{bx} \frac{4}{E_s \cdot d} = \sigma_{bx} \cdot k^2$$

where $k^2 = \frac{4}{E_s \cdot d}$

and $\sigma_{bx}$ is obtained from the equation of the bond stress-slip curve given by the experimental work.
7.1.3 **Equation of Bond Stress-Slip Curve**

Before the ordinary differential equation (O.D.E.) can be solved the equation of the bond stress-slip curve has to be found. To simplify the procedure the bond stress-slip curves (cf. Figs. 38, 50-52 and 59-61) were divided into three parts. The first two were represented linearly, the third being approximated to by a polynomial curve to the power three. This is shown in Fig. 103. The first part extends to the critical value, the second covers the transition period between the other two sections. The polynomial curve was arrived at by using the curve fitting facilities (54) on a computer.

Depending upon the bond stress-slip curve used, the first linear stage (that from zero slip to the critical value) has two possible equations. If the no stress during heating condition is considered then the curve begins at the origin whereas for the stressed specimens during heating the starting point is further up the ordinate axis. The equation for the second linear stage is always of the latter type of the two. Therefore the three equations making up the bond stress-slip curve are as follows

(a) \[ \sigma_b = m_1 \cdot \Delta \text{ or } \sigma_b = m_1 \cdot \Delta + c_1 \]

(b) \[ \sigma_b = m_2 \cdot \Delta + c_2 \]

(c) \[ \sigma_b = A_3 \Delta^3 + A_2 \Delta^2 + A_1 \Delta + A_0 \]

Therefore the O.D.E. has to be solved for three different cases.
7.1.4 Solution of O.D.E.

7.1.4.1 Case 1

\[ \sigma_b = m_1 \Delta \]

and from (6)

\[ \frac{d^2 \Delta}{dx^2} = m_1 \Delta \cdot k^2 \]

\[ = k_1^2 \Delta \]

where \( k_1^2 = k^2 \cdot m_1 = \frac{4m_1}{E_1 \cdot d} \)

for which a solution could be written in the form,

\[ \Delta = Ae^{k_1 x} + Be^{-k_1 x} \]

and

\[ \frac{d \Delta}{dx} = Ak_1 e^{k_1 x} - Bk_1 e^{-k_1 x} \]

Boundary conditions:

at \( x = 0 \), \( \Delta = \Delta_0 \)

\[ \frac{d \Delta}{dx} = \frac{\sigma_{sx_0}}{E_s} \]

Hence from (i) \( \Delta_0 = A + B \)

and from (ii) \( \left( \frac{d \Delta}{dx} \right)_0 = \frac{\sigma_{sx_0}}{E_s} = (A - B)k_1 \)

Solving (iii) and (iv) for \( A \) and \( B \) gives

\[ A = \frac{\Delta_0}{2} + \frac{\sigma_{sx_0}}{2E_s k_1} \]

\[ B = \frac{\Delta_0}{2} - \frac{\sigma_{sx_0}}{2E_s k_1} \]
Substituting $A$ and $B$ into (i) and (ii) gives

$$
\Delta_x = \left( \frac{\Delta_0}{2} + \frac{\sigma_{sx}}{2E} \right) e^{k_1 x} + \left( \frac{\Delta_0}{2} - \frac{\sigma_{sx}}{2E} \right) e^{-k_1 x} \quad - (7)
$$

$$
\sigma_{sx} = E k \left[ \left( \frac{\Delta_0}{2} + \frac{\sigma_{sx}}{2E} \right) e^{k_1 x} - \left( \frac{\Delta_0}{2} - \frac{\sigma_{sx}}{2E} \right) e^{-k_1 x} \right] \quad - (8)
$$

$$
\sigma_{tx} = m_1 \Delta_x \quad - (9)
$$

7.1.4.2 Case 2

This case refers to (b) referred to in 7.1.3.

$$
\sigma_b = m_2 \Delta + c_2
$$

and from (6),

$$
\frac{d^2 \Delta}{dx^2} = k^2 (m_2 \Delta + c_2)
$$

$$
= k^2 m_2 \Delta + k^2 c_2
$$

$$
= k_2 \Delta + k^2 c_2
$$

where $k_2^2 = k^2 m_2$

Thus

$$
\frac{d^2 \Delta}{dx^2} - k_2^2 \Delta = k^2 c_2
$$

for which a solution could be written in the form,

$$
\Delta = A e^{k_2 x} + B e^{-k_2 x} - \frac{c_2}{m_2} \quad - (v)
$$

$$
\frac{d\Delta}{dx} = A k_2 e^{k_2 x} - B k_2 e^{-k_2 x} \quad - (vi)
$$
Boundary conditions;

at \( x = 0 \), \( \Delta = \Delta_0 \)

\[
\frac{d\Delta}{dx} = \frac{\sigma_{sx0}}{E_s}
\]

Hence from (v)

\( \Delta_0 = A + B - \frac{c_2}{m_2} \) — (vii)

and from (vi)

\[
\frac{d\Delta}{dx} = \frac{\sigma_{sx0}}{E_s} = (A - B)k_2 - (viii)
\]

Solving (vii) and (viii) for \( A \) and \( B \) gives

\[
A = \frac{\Delta_0}{2} + \frac{c_2}{2m_2} + \frac{\sigma_{sx0}}{2E_s k_2}
\]

\[
B = \frac{\Delta_0}{2} + \frac{c_2}{2m_2} - \frac{\sigma_{sx0}}{2E_s k_2}
\]

Substituting \( A \) and \( B \) into (v) and (vi) gives

\[
\Delta_x = \left( \frac{\Delta_0}{2} + \frac{c_2}{2m_2} + \frac{\sigma_{sx0}}{2E_s k_2} \right) e^{k_2 x} + \left( \frac{\Delta_0}{2} + \frac{c_2}{2m_2} - \frac{\sigma_{sx0}}{2E_s k_2} \right) e^{-k_2 x} - \frac{c_2}{m_2} - (10)
\]

\[
\sigma_{sx} = E k \left[ \left( \frac{\Delta_0}{2} + \frac{c_2}{2m_2} + \frac{\sigma_{sx0}}{2E_s k_2} \right) e^{k_2 x} - \left( \frac{\Delta_0}{2} + \frac{c_2}{2m_2} - \frac{\sigma_{sx0}}{2E_s k_2} \right) e^{-k_2 x} \right] - (11)
\]

\[
\sigma_{bx} = m_2 \cdot \Delta_x + c_2 - (12)
\]

Also these expressions are relevant to the alternative equation in (a) of 7.1.3 above.
7.1.4.3 Case 3

This case refers to (c) referred to in 7.1.3.

\[ \sigma_b = A3\Delta^3 + A2\Delta^2 + A1\Delta + A0 \]

and from (6), \( \frac{d^2\Delta}{dx^2} = k^2(A3\Delta^3 + A2\Delta^2 + A1\Delta + A0) \)

First Integration

Reduce to a first order O.D.E.

let \( p = \Delta \); then \( \frac{d^2\Delta}{dx^2} = p \cdot \frac{dp}{d\Delta} \)

\[ \therefore p \cdot \frac{dp}{d\Delta} = k^2(A3\Delta^3 + A2\Delta^2 + A1\Delta + A0) \]

\[ \int p \cdot dp = k^2 \int (A3\Delta^3 + A2\Delta^2 + A1\Delta + A0) d\Delta \]

\[ p = \sqrt{2k} \left[ \frac{A3}{4} \Delta^4 + \frac{A2}{3} \Delta^3 + \frac{A1}{2} \Delta^2 + A0\Delta + A \right]^{\frac{3}{2}} \]

now \( p = \frac{d\Delta}{dx} = \frac{\sigma_{sx_0}}{E_s} \)

\[ \therefore \sigma_{sx_0} = \sqrt{2kE_s} \left[ \frac{A3}{4} \Delta^4 + \frac{A2}{3} \Delta^3 + \frac{A1}{2} \Delta^2 + A0\Delta + A \right]^{\frac{3}{2}} \]

From the boundary conditions

at \( x = C, \quad \Delta = \Delta_0 \)

\[ \sigma_{sx} = \sigma_{sx_0} \]

\[ \therefore A = \left( \frac{\sigma_{sx_0}}{\sqrt{2kE_s}} \right)^2 \left[ \frac{A3^2}{4} \Delta_0^4 + \frac{A2^2}{3} \Delta_0^3 + \frac{A1^2}{2} \Delta_0^2 + A0\Delta_0 \right] \]

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Second Integration

\[
\frac{d\Delta}{dx} = \sqrt{2k}\left(\frac{A_3}{4}\Delta^4 + \frac{A_2}{3}\Delta^3 + \frac{A_1}{2}\Delta^2 + AO\Delta + A\right)^{\frac{1}{2}}
\]

\[
\therefore \int \frac{d\Delta}{\left[\frac{A_3}{4}\Delta^4 + \frac{A_2}{3}\Delta^3 + \frac{A_1}{2}\Delta^2 + AO\Delta + A\right]^{\frac{1}{2}}} = \int \sqrt{2k}\,dx
\]

It was decided to solve the integral on the L.H.S. numerically by computer. For the moment let it's value be taken as Area 2.

\[
\therefore \text{Area 2} = \int \sqrt{2k}\,dx = \sqrt{2k}\Delta + B
\]

and within definite limits of \(\Delta\),

\[
x = \frac{\text{Area 2}}{\sqrt{2k}} \quad \quad \quad \quad (13)
\]

By solving numerically for different values of \(\Delta\), \(x\) can be calculated, therefore the function \(\Delta(x)\) is obtained.

For values of \(\Delta\) and the corresponding known values of \(x\) the \(\sigma_s(x)\) distribution can be calculated by using the expression,

\[
\sigma_{sx} = \sqrt{2k}\epsilon_s\left[\frac{A_3}{4}\Delta^4 + \frac{A_2}{3}\Delta^3 + \frac{A_1}{2}\Delta^2 + AO\Delta + A\right]^{\frac{1}{2}} \quad \quad (14)
\]

Similarly for values of \(\Delta\) and the corresponding known values of \(x\) the \(\sigma_b(x)\) distribution can also be obtained using the initial curve equation

\[
\sigma_b = A_3\Delta^3 + A_2\Delta^2 + A_1\Delta + AO \quad \quad (15)
\]

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7.1.5 **Initial Values**

To obtain the distributions along the bar length the initial conditions \( \Delta_0 \) and \( \sigma_{sx_0} \) had to be decided upon. If the case of a beam end is considered where the steel stress and end slip are both zero, the situation arises where the distribution curves cannot be evaluated. Therefore assumed initial conditions have to be chosen in order to begin the calculation.

The initial slip was, therefore, arbitrarily increased, to a value just greater than zero, of 0.0001mm while the steel stress at a distance of 1mm from the bar end was taken to be the initial steel stress at a value of \( x \) equal to 0.0. This steel stress was calculated as follows:

\[
\text{steel stress} = \frac{\text{force in steel}}{\text{area of steel}} = \frac{\text{bond stress} \times \text{bond area}}{\text{area of steel}} = \frac{\sigma_b(\Delta)^4}{d} \text{ per unit length}
\]

Consequently by using (6a)

\[
\sigma_{sx_0} = \frac{\sigma_b(\Delta_0)^4}{d} = \sigma_b(\Delta_0)^4 \cdot k^2 \cdot E_s
\]

These initial values of \( \Delta_0 = 0.0001\text{mm} \) and \( \sigma_{sx_0} = f(\Delta_0) \) were used for what has been termed the datum case for the distribution curves.

Apart from this datum case there are two particular situations to be investigated, the anchorage zone and the distributions in the region of cracks. As Rehm (33) points out the initial values are different for the two cases.

7.1.5.1 **Anchorage Zone**

In this case, theoretically, \( \sigma_{sx_0} \) is zero while \( \Delta_0 \) can vary according to the amount of slip that takes place.
The basic range of initial slip values used was 0.001, 0.002, 0.004, 0.008, 0.016, 0.032, 0.064, 0.128 and 0.256mm, while the zero value of \( \sigma_{sx_o} \) was replaced, as in the datum case above, by \( f(\Delta_o) \).

### 7.1.5.2 Crack Zone

In the region of cracks it is the initial steel stress values that vary while the initial slip value remains zero. Consider a length of beam between two cracks, see Fig. 104. At some point, the slip will be zero while the steel stress depends on the force being applied at the crack faces. If the steel stress at the two closely spaced adjacent cracks were equal then slip would be zero at the mid point between them.

The basic range of initial steel stresses used was 1.0, 2.0, 4.0, 8.0, 16.0, 32.0, 64.0, 128.0 and 256 N/mm\(^2\) with additional values being considered where necessary. Once again, for consistency, the zero initial slip value was replaced by a value of 0.0001mm. Such a wide range of initial stresses was chosen to cover the eventuality of cracks developing at different stress levels.

### 7.1.6 Computer Program

A Fortran computer program was written to solve the numerical integration and also give the slip, steel stress and bond stress distributions (i.e. \( \Delta(x) \), \( \sigma_s(x) \) and \( \sigma_b(x) \)) along a length of bar. The basic program is given in appendix 7. There are slight variations for the two situations that were investigated i.e., anchorage and cracking zones. The program given is for the former case while the adjustments for the latter are listed at the side.

Points to note include the three subroutines, two of which correspond to the first linear stage. One evaluates
points along the curve while the other calculates the final values for this stage, which in turn become the initial values for the second linear stage calculated by the third subroutine. The numerical integration was solved using Simpson's rule and loops were incorporated to obtain results for a range of initial conditions i.e., $\Delta_0$ varies for the anchorage zone while $\sigma_{sx_0}$ varies in the region of cracks.

7.2 DISTRIBUTIONS FOR DATUM INITIAL CONDITIONS

\[ \Delta_0 = 0.0001\text{mm}; \sigma_{sx_0} = f(\Delta_0) \]

7.2.1 Further Considerations of Initial Values

$(\Delta_0$ and $\sigma_{sx_0}$)

The application of the above initial values is straightforward for the no stress conditions. However for those specimens subjected to load during heating the situation is more complex. The reason being that the bond stress-slip curves were obtained during loading to failure after the heating cycle was completed. This means that for the stressed specimens slip occurred before loading to failure took place. Therefore the slip occurring during the heating cycle has to be taken into account when estimating the initial slip value to be used for obtaining the distribution curves.

7.2.1.1 Estimation of Slip During Heating Cycle

The apparent slip that took place during the heating cycle (heating up and cooling down) for each temperature can be obtained from Figs. 35 and 37 for the stressed specimens. This apparent slip movement is made up of two components. Firstly the actual slip between the steel and concrete and secondly any creep that occurred during the heating cycle. It was not possible to separate the components experimentally however it was possible to make an estimate of each component from the known results. This is achieved from consideration of the 150 and 300°C temperature levels as follows.
Consider the residual no-stress specimens. For a bond stress of 3.70 N/mm² a slip of 0.0062 and 0.0148mm took place for temperatures of 150 and 300°C respectively (from Fig. 52). By comparison the residual specimens stressed throughout the heating cycle gave an apparent slip value considerably larger after the heating cycle was completed i.e., 0.0725 and 0.150mm for 150 and 300°C respectively (see Figs. 35 and 37).

However beyond the bond stress of 3.70 N/mm² the bond stress-slip curves are remarkably similar for both sets of specimens. They are not identical as seen in 5.1.3.1, but this difference is not sufficient to account for such a greatly increased additional slip for the stressed conditions. Indeed slips of the same order of magnitude for both stressed and no-stressed conditions are to be expected for bond stresses up to 3.70 N/mm² in the 20-450°C temperature range. This statement is based on the fact that the critical value was not exceeded for temperatures of 150 to 450°C while being stressed during the heating cycle. This means that the slip was still dependent on the movement of the mortar immediately beneath the ribs which had not exceeded its maximum compressive strength. Therefore the slip, being within the critical value range, was related to the concrete strain beneath the ribs which was in turn related to Young's modulus for concrete. To further confirm this deduction the E values were calculated for both the stressed (using the apparent slip) and the no-stress (using the actual slip at a bond stress of 3.70 N/mm²) conditions and compared with accepted published values. The calculation is given in appendix 8. For the no-stressed condition the E values at 150 and 300°C were 50% and 21% of the 20°C value which admittedly gives a greater reduction than shown in Fig. 5. However it is of the same order of magnitude where variations can be explained by differences in the type of concrete and test conditions. In contrast to this the apparent slip for the stressed conditions gives values of Young's modulus that are even
more significantly reduced (viz. 4.28% and 2.07% of the ambient value for 150 and 300°C respectively), when in fact the applied loading during heating would be expected to increase Young's modulus compared to the no-stress condition(7).

Therefore it is concluded that the great increase in apparent slip for the stressed specimens is not real slip but attributable to the effects of creep.

From this it can be seen that the actual slip of the stressed specimens after the heating cycle is finished is approximately equivalent to that for the no-stress specimens at the bond stress of 3.70 N/mm². It will not be exactly the same as stated in 5.1.3.1.1 where the differences in the results for the various test conditions are explained.

When estimating the initial slip value for stressed specimens these slight variations can be accounted for by utilising the initial gradient of the bond stress-slip curve. The bond stress-slip curve is approximated to a linear curve between the origin and the critical value (cf. Fig. 103) hence knowing the gradient between a bond stress of 3.70 N/mm² and the critical value and by extending it backwards to the origin an estimate of the initial slip can be made.

This procedure is valid only for temperatures up to 450°C i.e., where the critical value is not exceeded during the heating cycle. For the higher temperatures of 600 and 750°C it is not possible to deduce the breakdown of the apparent slip into actual slip and creep as the critical value was exceeded during the heating up period. Hence these temperature curves are not plotted for the stressed conditions in Figs. 105-107.
7.2.2 Comments on Distribution Curves

7.2.2.1 General

The distributions for the four test conditions (a)-(d) (i.e., stressed, hot; unstressed, hot; stressed, residual; unstressed, residual) and datum initial conditions are given in Figs. 105-107. The slip increase with distance x which in turn produces a corresponding increase in steel stress. The bond stress is dependent on the rate of change of the steel stress with length at any particular point. As with the bond stress-slip curves the critical value influences the respective distribution curves. This is seen most clearly for bond stress distributions Fig. 107. Immediately before the critical value the bond stress increases rapidly then the rate of increase noticeably decreases after the critical value is passed. The effect of the initial value on the distributions can be seen from Figs. 105-107 as the distributions for a bond stress-slip curve with no critical value are also plotted (i.e., eqn. of curve is \( \sigma_b = m_1 \Delta \)). Therefore after the critical value the rate of increase in the slip, steel and bond stress distributions all begin to decline.

The figures show that for the hot and residual conditions there is little difference in the distributions for temperatures up to 300°C. At 450°C there is a slight increase in the variation which becomes appreciable at 600 and 750°C following the pattern of the bond stress-slip curves. The bond stress distribution curves show up the differences more clearly than the other two cases.

7.2.2.2 Stressed and No-stressed Conditions

The most obvious difference in the distributions mentioned above is that between the stressed and no-stressed conditions.
7.2.2.2.1 No-stressed condition:— This difference stems from the effect the initial part of the bond stress-slip curve (i.e., $0 \leq \sigma_B \leq 3.70$ N/mm$^2$) has on the no-stress distributions at the various temperature levels. As the gradient decreases with an increase in temperature over the initial portion of a stress-slip curve so the distance $x$ required to attain the same stress levels becomes greater. This implies that a greater bond length is required to resist the same force for constant initial values of $\Delta_0$ and $\sigma_{sx0}$ at each temperature. Therefore the steeper the distribution the more favourable the bond performance.

7.2.2.2 Stressed condition:— Unlike the no-stress condition the distributions for the stressed specimens appear to be much less affected by the heat treatment. However this is an apparent difference only. It was established in 7.2.1 that the stressed distribution curves have to be plotted for initial values of slip closely corresponding to the slip at $\sigma_B = 3.70$ N/mm$^2$ of the no stressed specimens. It follows that the stressed distributions are in effect equivalent to but not identical with the portion of the complete distribution above $\sigma_B = 3.70$ N/mm$^2$ obtained from the no-stress condition. The one difference being that for the steady state stress during heating case the initial steel stress value is a function of the initial slip value, e.g., 0.0031 mm (cf. Fig. 106(b)), whereas for the situation of no steady state stress during heating the steel at a distance $x$ where a slip of 0.0031 mm has occurred is somewhat higher (e.g., from Figs. 105-107 for 20°C, if $\sigma_B = 3.70$ N/mm$^2$ then $\Delta = 0.0031$ mm and $\sigma_s = 31$ N/mm$^2$ compared with $\sigma_s \approx$ zero for $\Delta_0 = 0.0031$ mm).

Therefore by starting the distributions at the bond stress of 3.70 N/mm$^2$ the initial part of the stress-slip curve is ommitted hence the similarity in the curves, especially for the slip and steel stress, beyond this point and for temperatures up to 300°C is highlighted. As the temperature increases so a decline in the distributions is
seen. This is especially so at 600 and 750°C but also occurs for 450°C as the steel and bond stress distributions show. When these curves are compared with the load cycling curves of Figs. 69 and 70 it is noticed that for temperatures above 20°C but below 400°C the results are close to each other but by the time 400°C is reached a further deterioration takes place. Hence another indication is obtained that between 250 and 400°C a critical temperature is attained that causes a further reduction in the bond performance.

7.2.2.3 Comparison of Different Covers

Figs. 108-110 show the distributions for the four different covers tested. These distribution curves are for the stressed, residual test condition results and were plotted from a zero bond stress datum by assuming that the gradient of the bond stress-slip curve between the bond stress of 3.70 N/mm² and the critical value is maintained between the 0.00 and 3.70 N/mm² bond stress values. The datum initial conditions of $\Delta_0 = 0.0001$mm and $\sigma_{ex_0} = f(\Delta_0)$ were used.

On the whole the different covers give very similar results for the lower temperatures up to 300°C. The early splitting failure of the smaller covers is responsible for the lower maximum value of slip, steel and bond stress being achieved in these cases compared with that for the larger covers. The application of temperature does not alter this as the similarity between the ambient and 150/300°C curves makes clear. Therefore it can be said that the different covers had little or no effect on the distributions for temperatures up to 300°C. For temperatures beyond this point there was considerable change due to the varying cover sizes. The 32mm cover curves at 450°C emphasise this point as does the fact that for the 25mm cover the bar pulled through before 450°C was reached. These comments apply to the stressed, residual condition only.
7.3 DISTRIBUTIONS FOR ANCHORAGE ZONE

7.3.1 General Effects of Increased Slip

Figs. 111-116 give results for various assumed end slips. The no-stressed, residual condition for the 55mm cover is presented in the form of complete distribution curves. The distributions follow an expected pattern typified by Fig. 111(a)-(c). At low slip values the slip and steel stress curves have a parabolic shape, the bond stress follows this pattern up to the critical value beyond which the rate of increase declines suddenly. However as the end slip increases so the parabolic shape tends more towards a linear curve. The steel stress increases from zero at the bar end while the bond stress tends to become more uniform throughout the bar length. In practical terms it approaches finally to the point where the bar is being pulled through giving no increase in bond stress which is uniform along its length. The three distributions are shown for 20°C only, while the steel stress curve is given for each temperature since for the anchorage zone the steel stress curve is the one of most interest.

7.3.2 Effect of Heat on the Anchorage Length

The bond length required is related to the force and hence the stress in the bar, therefore a sufficient length is needed to ensure that the necessary level of stress can be achieved. From Figs. 111-116 the effect of the temperature on the distributions is shown to be considerable e.g., for an end slip of 0.001mm the distance at which a steel stress of 100 N/mm² is achieved is 92, 140, 228, 404, 1025 and 1992mm for temperatures of 20, 150, 300, 450, 600 and 750°C respectively. According to this pattern at 300°C a bond length of almost two and a half times that at ambient conditions is required. If another example is taken, this time for an end slip of 0.0046mm (i.e., the critical slip value at ambient conditions) and a steel stress of 100 N/mm²
the distances are 62, 90, 150, 280, 760 and 1525mm respectively. Again at 300°C the increase in bond length required is of the order of two and a half times the 20°C value, which is a considerable increase.

In the above examples the assumption has been made that for such a comparison, a uniform end slip for all temperatures has to be used. However this seems unlikely to be a realistic assessment of the situation because of the effect heat has on the material properties of concrete. In the review, chapter 1, it was noted that at high temperatures the stiffness of concrete decreases and it becomes a slightly less brittle material enabling more movement to be accommodated within it. This is reflected in the decreased gradient of the bond stress-slip curves up to the critical value as the temperature increases (see Fig. 39). In view of this a greater end slip can be sustained at elevated temperature compared with that at ambient before the bond stress reaches the same level. Then the question arises, what is the magnitude of slip at the end of the anchorage zone that denotes an equivalent level of bond breakdown for different temperature levels? It seems reasonable to assume that the one point that this information is available is at the much mentioned critical value. To add to this it could also be said that the slip at the critical value at the bar end marks the final failure of any useful bond resistance of the anchorage zone. In other words the critical slip at the bar end can be considered to have occurred when the concrete immediately beneath the ribs at that point exceeds its maximum strength. As the bond stress distribution curves show it is at this value that the bond stress moves towards a uniform value over the bar length, with the steel stress tending more to a linear curve.

Therefore it is suggested that a more realistic basis for comparing the effect of temperature on the bond length should not be one of a constant end slip criteria for all temperatures but rather should be based upon the variation in slip at the critical value for different temperatures. The
relevant end slip criteria for the 20–750°C temperature range is 0.0046, 0.0084, 0.0207, 0.0322, 0.07 and 0.1mm respectively. In this case the respective bond lengths required to sustain a steel stress of 100 N/mm² are 62, 72, 75, 118, 228 and 472mm. This time the increased bond length for 300°C is around 1.3 times that for the ambient temperature level.

Fig. 117 shows these points plotted along with the corresponding results for steel stresses of 200 and 300 N/mm². The familiar pattern emerged of a decline in bond performance on initial heating, which remained relatively constant up to the 250°C interface temperature after which a further deterioration occurred.

7.4 DISTRIBUTIONS FOR CRACK ZONE

7.4.1 General Effects of a Varied Initial Steel Stress

Altering the initial conditions so that the initial slip remained constant, while the initial steel stress was varied arbitrarily (from 1-256 in increasing powers of two with additional values where required) changed the form of the distribution curves, as seen in Fig. 118. The slip and steel stress distributions are both parabolic, tending to a linear curve while the bond stress curves, far from tending to a constant, show an increasing gradient as the initial steel stress increases. This is consistent with the situation in practice where the higher steel stress gives a greater strain and hence slip over the bar length resulting in the bond stresses increasing at a greater rate. Once again the critical value is easily picked out with the decline in the rate of increase of the bond stress. In a manner similar to that for the anchorage condition the three distributions are shown only for 20°C with the steel stress case only given for the elevated temperature levels (see Figs. 118-123).
7.4.2 Effect of Heat on Crack Spacing

7.4.2.1 General

Rehm\(^{33}\) outlines a procedure for calculating the minimum crack spacing at ambient conditions. This is modified here to take account of the high temperatures involved.

Fig. 124(a) shows the situation at a tensile crack in (say) a beam. At the crack the stress in the concrete, \(\sigma_c\), is zero and the steel resists the tensile force present. As the distance from the crack increases the concrete begins to contribute permitting a decrease in the level of the steel stress, \(\sigma_s\). This continues until at some distance along the bar the pre-crack values of \(\sigma_c\), \(\sigma_s\) and the bond stress, \(\sigma_b\), and slip, \(\Delta\), are attained. At this cross section, XX, a second crack could develop resulting from increased loading since \(\sigma_c\) at XX would have a peak value in a decreasing bending moment field i.e., one in which the bending moment diagram decreases towards the support. To investigate the crack spacing an estimate of the distance from the first crack to where the tensile stress in the concrete obtains its limiting value, \(\sigma_{ct}\), has to be made. If the change in the concrete tensile stress is considered in terms of the change in the steel stress then from the steel stress distribution curves an estimate of the distance involved can be made.

In order to do this simplifying assumptions have to be made concerning the distribution of the tensile stresses and the size of the loaded area in the concrete. Both of these are complex matters. However, it is assumed here that for say a 55mm all round cover the entire cross section of the concrete is contributing to the tensile resistance and hence the tensile stresses in the concrete are proportional to the tensile force involved under elastic conditions. Assuming that negligible slip occurs prior to the initiation of the first crack, then after the first crack but prior to the appearance of the second one it could be argued, with the aid of Fig. 124(a),
that the section at which \( \Delta = 0 \) occurs, should coincide with the section at which the influence of the first crack on the general state of stress disappears. Therefore from Fig. 124(a) the change in the steel stress at some distance \( x_1 \) from the crack is related to the increase in the concrete stress in the following way.

\[
\left( \sigma_{sx_1} - \sigma_{sx_0} \right)A_s = (\sigma_{ct} - 0)A_c \\
\Rightarrow \left( \sigma_{sx_1} - \sigma_{sx_0} \right)A_s = \sigma_{ct}A_c - (16)
\]

Before this relationship can be used the value of \( \sigma_{sx_0} \) must be known. In other words the initial values for slip and steel stress have to be established.

Referring again to Fig. 124(a). Maximum slip occurs at the crack and then decreases in value until it is equal to zero at the point where the concrete is fully participating in resisting the tensile stress. At such a point the strain in the steel and concrete is the same, and so

\[
\sigma_{sx_0} = m \sigma_{ct} - (17)
\]

where \( m \) is the modular ratio. Therefore the initial conditions are \( \Delta_0 = 0.0001 \) and \( \sigma_{sx_0} = m \sigma_{ct} \) where \( \Delta_0 = 0.00 \) is replaced by \( \Delta_0 = 0.0001 \)mm for consistency with 7.1.5.

7.4.292 Tensile Strength of Concrete

As equation 17 makes clear it is necessary to know the tensile strength of the concrete before the crack spacing can be calculated. The effect of heat on the maximum residual tensile strength was estimated from the indirect splitting tests carried out on the bond test specimens. It is difficult to arrive at a totally accurate value due to the characteristics of the test specimens used i.e., a concrete
cylinder with a hole running through the centre further complicated by the steel bar being embedded over only 32mm of it's length.

To estimate the tensile strength the indirect tensile strength test value of

\[ \sigma_{ct} = \frac{2P}{\pi D l} \]

was modified to

\[ \sigma_{ct} = \frac{2P}{\pi(D - d_1)l} \]

where \( D = \) overall diameter
\( d_1 = \) diameter of the central hole
\( l = \) length or height of cylinder

It is realised that this is only an estimate. The actual stresses over the cross-section will vary from a very high concentration at the hole to a value which is equivalent to that of a normal solid cylinder towards the edge (cf. Fig. 125 and ref. 55). Therefore the effective diameter was reduced in an attempt to arrive at a mean value. It is likely that even with this modification the value obtained will be an underestimate of the actual value. However this could be compensated for as the indirect tensile test gives values that are higher than the direct tensile strength of the concrete. Neville\(^{(56)}\) states that this difference is of the order of 5-12 percent. Following this procedure a value of 2.0 N/mm\(^2\) was obtained for the concrete tensile strength at ambient temperature (see Table 5).

7.4.2.3 Young's Modulus for Concrete in Tension

The change in the \( E \) value for concrete in tension after heat treatment also has to be known if equation 17 is to be of use. Using a formula put forward by Madu\(^{(57)}\) the value at
ambient temperatures can be estimated

e.g., \[ E \text{ value of concrete in tension at zero stress } = 9.5 + 2.5\sigma_{ct} \]
\[ \text{and ambient temperature } = 14500.0 \text{ kN/mm}^2 \]

where \( \sigma_{ct} \) is expressed in N/mm\(^2\).

However the effect of temperature on the stress-strain curve for concrete in tension is not known. There appears to be no published work on this particular characteristic of concrete in the elevated or residual condition. Therefore it is not possible to accurately assess the value of Young's modulus for this situation. Due to the lack of information, for the purposes of this investigation, the effect of heat on the \( E \) value for concrete in tension has been assumed to follow the pattern of that for concrete in compression. (cf. Fig. 5). This may not be an accurate assumption to make, but knowing that concrete becomes more ductile after heat treatment it seems reasonable to expect a decrease in Young's modulus for both tension and compression. Therefore although the two might not be affected in exactly the same way (cf. the difference in the tensile and compressive strengths in Fig. 83) a decrease in both can be expected. Hence the results in compression are used as a guide to estimate what those in tension could be. This is further complicated as the \( E \) values of Fig. 5 are for the elevated condition; whereas it is the residual case that is under consideration here. This could cause an additional decrease in \( E \) value to occur similar to the behaviour of the compressive strength for the two conditions. Therefore the lowest values from Fig. 5 have been used as a basis for computing the changes in the tensile value of \( E \) for the concrete in the residual condition. These are listed in Table 5 along with \( m, \sigma_{ct} \) and \( \sigma_{sx0} \) for the associated temperature levels.
7.4.2.4 Procedure for Estimating Crack Spacing

To illustrate how the crack spacing is estimated, the 20°C temperature will be considered for a 55mm cover in the no-stress, residual condition. The bar diameter is 16mm.

Initial values:

\[ \Delta_0 = 0.0001, \quad \sigma_{sx_0} = m \cdot \sigma_{ct} \]

Now from Table 5 \( \sigma_{ct} = 2.0 \text{ N/mm}^2 \) and \( m = 13.795 \), hence \( \sigma_{sx_0} = 27.59 \text{ N/mm}^2 \).

The difference in steel stress to produce the maximum tensile strength in the concrete is obtained from equation (16) as,

\[ (\sigma_{sx_1} - \sigma_{sx_0}) = \sigma_{ct} \cdot \Delta_c \]

\[ = 2.0 \times 61.0 \]

\[ = 122.0 \text{ N/mm}^2 \]

\[ \therefore \sigma_{sx_1} = 27.59 + 122.0 \]

\[ = 149.59 \text{ N/mm}^2 \]

From Fig. 118(b) for an initial steel stress of 27.59 N/mm² a distance of 86mm is needed to produce the difference in steel stress required, which is the minimum possible spacing for the next crack.

Now consider the situation shown in Fig. 124(b). The minimum crack spacing corresponds to the distance required for the concrete tensile stress to reach its failure level. However for a section of constant bending moment further cracking can occur at any section from this point onwards. If it is arbitrarily assumed that the second crack occurs when \( \sigma_{ct} \) first reaches its failure level i.e., giving a crack spacing of 86mm for the example above, then any further
cracking that occurs between these two existing cracks can be taken to develop mid-way between them i.e., within a distance of 43.0 mm. Fig. 118(b) shows that the initial steel stress for such an intermediate crack to develop is very high i.e., greater than 450.0 N/mm². Therefore at 20°C 89 mm is the minimum crack spacing within the working range of the steel stress.

If another example is considered e.g., the 450°C case, the situation arises where further cracking between the two original cracks does take place.

Initial values:

\[ \Delta_0 = 0.00019 \text{m}, \quad \sigma_{sx0} = m \cdot \sigma_{ct} \]

Now from table 5 \( \sigma_{ct} = 1.14 \text{ N/mm}^2 \) and \( m = 28.74 \), hence \( \sigma_{sx0} = 32.76 \text{ N/mm}^2 \).

(Note: E for steel remains 200000 N/mm² as the steel recovers most of its original strength properties after cooling.)

Steel stress difference to produce cracking:

\[ (\sigma_{sx1} - \sigma_{sx0}) = \frac{A_c}{A_s} \]

\[ = 1.14 \times 61.0 \]

\[ = 69.54 \text{ N/mm}^2 \]

\[ \therefore \sigma_{sx1} = 32.76 + 69.54 \]

\[ = 102.3 \text{ N/mm}^2 \]

From Fig. 121 the minimum crack spacing is 166 mm. For half that distance an initial steel stress of approximately 260 N/mm² gives the difference of 69.54 N/mm² required to cause further cracking before yielding of the steel takes place. Therefore as the load is increased additional cracking occurs.
Using this procedure an estimate of the relationship between the crack spacing and temperature can be obtained. The procedure is limited to sections of constant bending moment, as it is for this condition only that a further crack can be taken to occur half way between the first two cracked sections. Also it is difficult to justify for constant bending conditions, the assumption that the second crack occurs when \( \sigma_{ct} \) is first attained some distance from the initial crack. By comparison, for the varying bending moment case of Fig. 124(a) it is highly likely that the second crack would occur at the corresponding section XX, but when there is a constant bending moment the situation is not so clear cut. However the purpose here is to examine the effect of heat treatment on the crack spacing, therefore constant conditions must be considered for each temperature level. By assuming that the second crack occurs when \( \sigma_{ct} \) first attains its maximum value i.e., at section XX, such a constant condition for each temperature is obtained. Hence the crack spacing-temperature relationship for one particular bending moment condition can be established. However, it is possible that the temperature would have a similar effect on cracking in zones just outside and adjacent to that of uniform bending. The calculated results were compared with those for actual beam tests using a central zone extending slightly beyond that of uniform bending, as set down in the next section. For the calculated results both the strength and the Young's modulus values for concrete in tension were obtained as stated in 7.4.2.2 and 7.4.2.3, while the area of concrete assumed to be contributing to the resistance of the tensile stresses was that contained by a square block surrounding the bar whose dimensions were twice the cover plus the bar diameter as shown in Fig. 126.
7.5 COMPARISON OF RESULTS WITH BEAM TESTS

The anchorage and crack spacing distributions were considered for the no-stress, residual condition as small beam tests were carried out by M. R. Khan for this case. Therefore an opportunity was provided to compare the calculated results mentioned above with the actual crack spacing-temperature relationship obtained in some beam tests. The beams used were approximately 1m long, had covers of 20, 30, and 40mm and were reinforced with an 8mm diameter deformed bar. The concrete mix was the same as that used for the bond tests, and the beams were tested under four point loading.

7.5.1 Crack Spacing

The crack spacing-temperature relationship for the actual beams is shown in Fig. 127. It was obtained for the flexural cracking over the region of constant bending moment and slightly beyond, but not including the bond-shear cracking nearer the supports. Hence a closer approximation to the constant bending moment condition of the calculated results was obtained. The general shape of the curve shows a decrease in the spacing as the temperature rises to 200/300°C at which level a change occurs and the spacing increases up to 400/500°C. Each point represents the results from one beam test.

For comparison with these results the calculated values were obtained for four different test cases. The first was the 55mm cover in the residual, no-stress condition with a 16mm diameter bar, thus utilising the bond stress-slip results obtained. However when comparing with the beam tests a difference in parameters has to be taken into account i.e., the 20, 30 and 40mm covers and 8mm bar of the beams as opposed to the 55mm cover and 16mm bar of the pull-out tests. Therefore to make a fairer comparison the bond stress-slip curves for the three other bond test covers were also
considered i.e. the 25, 32 and 46mm covers respectively. When using results of the three bond test covers to calculate the crack spacing for the respective beam tests, three variations in the two types of test are apparent. Firstly there is the difference in the size of the covers themselves (e.g., 46 as opposed to 40 etc.). However it has already been noted from Figs. 108-110 that at the lower temperatures the cover has little effect on the distribution curves, causing any error from this inconsistency to be negligible. The second difference is that the bond results at these three covers were obtained under the residual, stressed condition as opposed to the residual, unstressed case of the beams. However the difference this causes at the lower temperatures is also relatively small (see 5.1.3.1). The greatest error is introduced by the difference in the bar diameter. The steel stress distributions for the three bond test covers were calculated on the basis of an 8mm diameter bar, whereas the bond stress-slip curves used were those for the 16mm diameter. In reality the bond stress-slip curves for the 8mm bar will differ from those obtained for the 16mm bar. However this is the closest possible approach to the beam test conditions that can be made with the available pull-out data. A comparison was made on this basis to see if any meaningful conclusions could be drawn.

The calculated results are given in Fig. 127. As can be seen the theoretical values do not bear much resemblance to the actual ones. The only similarity being the overall decrease in magnitude of the crack spacing with the corresponding decrease in cover size.

In an attempt to improve upon the calculated values the procedure was repeated, this time making an allowance for the contribution the concrete strain makes towards the slip value.

7.5.1.1 Incorporating Concrete Strains

The basic relationship as given in section 7.1.2 is

116
\[
\frac{d\Delta}{dx} = \varepsilon_s - \varepsilon_c = \frac{\sigma_s}{E_s} - \frac{\sigma_c}{E_c}
\]

Now from Fig. 124(a) and section 7.4.2.1 for \(0 < x < x_1\),

\[
(\sigma_{sx} - \sigma_{sx_0})A_s = (\sigma_{cx} - \sigma_{cx_0})A_c
\]

\[
\therefore \sigma_{cx_0} = \sigma_{cx} - \frac{(\sigma_{sx} - \sigma_{sx_0})A_s}{A_c}
\]

\[
= \sigma_{cx} + (\sigma_{sx} - \sigma_{sx_0})\mu
\]

where \(\mu = \frac{A_s}{A_c}\)

For a crack at \(x = x_1\), \(\sigma_{cx} = \sigma_{cx_1} = 0\) and \(\sigma_{sx} = \sigma_{sx_1}\)

\[
\therefore \sigma_{cx_0} = (\sigma_{sx_1} - \sigma_{sx_0})\mu
\]

and so from equations (19) and (20),

\[
\sigma_{cx} = (\sigma_{sx_1} - \sigma_{sx})\mu
\]

Substituting equation (21) into (18) gives,

\[
\frac{d\Delta}{dx} = \frac{\sigma_{sx}}{E_s} - \frac{(\sigma_{sx_1} - \sigma_{sx})\mu}{E_c}
\]

and rearranging

\[
\frac{d\Delta}{dx} = \frac{\sigma_{sx}}{E_s}(1 + m\mu) - \frac{\sigma_{sx_1}}{E_c}\mu
\]

where \(\sigma_{sx_1}\) is the steel stress at the crack face.
By utilising equation (22) and following a similar procedure to that mapped out in 7.1.4, distribution curves (see Figs. 128-131) can be obtained making an allowance for the concrete strains. When following this through the initial values to be used have to be decided upon. For the theoretical initial values of steel stress equivalent to \( m \sigma_{ct} \) and slip equal to zero, the distribution curves are indeterminate. A situation is reached similar to that of \( \sigma_{sx} = 0.00 \) and \( \Delta_0 = 0.00 \) for the anchorage zone case using steel strains only, as discussed in 7.1.5. Therefore to obtain a distribution curve a slight increase in one or other of these values has to be assumed. In an attempt to keep these values consistent throughout, an initial slip of 0.0001mm was once again used.

From Figs. 128-131 and 118-121 the distribution curves with and without taking account of the concrete strains, are seen to be very similar for high initial steel stress values. However as the initial steel stress approaches a value equivalent to \( m \sigma_{ct} \) so the distributions allowing for concrete strains extend over a greater bar distance \( x \). This occurs at low steel stress and hence strain levels. In this situation the concrete strains are sufficient to be relatively high when compared with the corresponding steel strains, whereas as the steel strain increases the relative effect of the concrete strain diminishes. Hence at low stress levels the concrete contributes a greater proportion of the resistance than at high stress levels.

Using these new steel distribution curves the effect of heating on the crack spacing was again considered. The actual and calculated results for the three beam sizes (covers) are shown in Figs. 132-134. On the whole the general shape of all the curves is similar, the one exception being the 300°C calculated value for the 40mm cover beams, which is higher than would be expected. The calculated results for the 30mm cover beams basically follow the pattern of the actual results. For the 20mm cover beam the basic trough in the crack
spacing-temperature curve is apparent, although in this case the magnitude of the calculated results are considerably lower than the actual values. The 450°C furnace temperature point is not available as the 25mm cover bond specimens pulled through before this temperature level was reached (see Fig. 62), therefore the bond stress-slip and hence the steel stress distributions were unable to be calculated.

On the whole by making an allowance for the concrete strains the results obtained are much more encouraging. The required trend of the smaller covers giving smaller values of crack spacing is apparent. The crack spacing itself is of the right order of magnitude and also the effect of heat on the spacing is able to be established up to 450/500°C.

7.5.2 Anchorage Zone

For temperatures above 500°C the calculated crack spacing results would appear to no longer be valid for comparisons with the actual beam results. The reason is as follows. For the beams heated to temperatures up to 500°C, failure and cracking occurred predominantly within the central portion of the beam. Although the inclined cracks caused by bond-shear splitting could also be identified at these temperatures, on the whole a flexural type failure occurred (see Fig. 135(a)). However at temperatures of 600°C and above the failure mode changed. The main cracking occurred in the direction normal to that of the principal tensile stresses indicating that, at these temperatures, the beam was likely to have been acting along the lines of a tied arch (see Fig. 135(b)). This would be the result if the reduction in the bond performance was such that the required stress in the steel could not be achieved. To utilise this tied arch action to the full adequate anchorage would be required e.g., hooks or bends in the reinforcement. In the beams considered here sufficient anchorage was not available. Hence the splitting extended from the lower face or soffit of the beam at or near the supports and spread upwards towards the internal loading.
points (see Fig. 135(b)), showing that slip had occurred in the anchorage zone at the bar end.

Hence this change of failure within the concrete beam would seem to have been caused by a reduction in the bond, resulting in a predominantly anchorage zone failure. Therefore at the higher temperatures it was the anchorage zone that was critical as regards failure of the beams under consideration. This can be appreciated by reference to Fig. 117 where the required anchorage length increases dramatically at these high temperatures.

However cracking did occur also in the area of maximum bending moment as Fig. 135(b) shows, although it was to a lesser extent than the lower temperature cracking examined already. From the distribution curves the crack spacings for temperatures of 600°C and above would be expected to be greater than those for lower temperature levels. However this was not the case for the beams tested. The reason for this would seem to be as follows. At the lower temperatures (i.e. below 600°C) the change in steel stress over a distance x could cause the maximum tensile strength of the concrete to be attained resulting in the development of a crack, which in turn would cause bond slip. Whereas at temperatures of 600°C and above the bond was so reduced that a far greater distance could be anticipated for the development of the necessary difference in steel stress to produce a crack (i.e., 180mm; see Fig. 122 and compare the calculated results given in Fig. 127). However in this case the bond stress reduction was probably such as to allow slip within the beam to take place, which amounted to the steel and concrete acting more as two separate units rather than one composite whole. Thus in this case it would be the fact that the concrete was acting more as an independent beam, that caused cracking to occur in the region of maximum bending moment, at a smaller spacing than was predicted using the procedure of 7.4.2.4. Consequently at the lower temperatures it could be considered that cracks had occurred which resulted in slip while at the higher temperatures slip
took place which resulted in cracking.

7.5.3 Summary

From the bond stress-slip curves the distribution curves were obtained for the anchorage zone and crack spacing cases. The effect of heat upon both of these conditions was then studied and compared with the results of tests carried out on small beam specimens by M.R. Khan.

This comparison showed that when the concrete strains are taken into account a reasonable estimate of the crack spacing-temperature relationship could be made. Also the theoretical results of the effects on the anchorage zone of heat were backed up by the anchorage failure of the high temperature beams. The comparison between the calculated and actual results was therefore reasonably good considering the limitations of the theoretical approach (e.g., the difference in parameters of the beam and bond tests, the assumptions on the area and distribution characteristics of the tensile stresses within the concrete etc.).

Having established that the theoretical approach of applying bond stress-slip results from a short bond length to distributions over a larger bar length gives results comparable to those in practice, it then follows that the bond test specimen of a small differential bond length could give results that are of practical use and significance. Therefore not only is the theoretical approach seen to give reasonable results but also the bond test procedure gives valid results that can be applied to situations in practice.

7.6 DESIGN CONSIDERATIONS

The processes of designing for the reinstatement of fire damaged concrete members and rational design for fire resistance were reviewed in chapter 1, sections 1.3.2 and 1.4. The application of the bond results to these situations will
7.6.1 Designing For Reinstatement

The known reduction in strength of the material properties is used to assess the ability of the fire damaged member to resist the applied loading involved. This can be achieved by applying fire-damage factors to the concrete and reinforcement strength properties subjected to heat treatment. Fig. 7 shows the fire-damage factor for reinforcement put forward in ref. 11. A similar factor was suggested for concrete i.e., a factor of 0.85 for concrete heated to between 100 and 300°C, and unity for temperatures below 100°C. If the concrete is heated to above 300°C it is considered unsuitable for reuse and removed. One further fire-damage factor was suggested, that of 0.7, to be applied to the steel reinforcement embedded within the concrete in the 100-300°C zone just mentioned. It has been stated that,

'for steel reinforcement in this zone, a damage factor should be assessed to allow for the possibility of reduced bond or anchorage; there is very little evidence to quantify this loss (if any), but a damage factor of 0.7 is suggested.'

From the results obtained in this work it is possible to comment on the reduction in the performance of a member due to the effect of the heat treatment on the bond.

7.6.1.1 Temperatures of 300°C and Above

It has been noted that the bond performance was strongly related to that of the concrete compressive strength as Figs. 58, 63, and 83 show. Therefore as concrete, which has been heated to temperatures in excess of 300°C, is not considered suitable for the reinstatement process it follows that the residual bond following heating to such temperatures would not be adequate for reuse either. The results of the test programme conducted in this work indicated that this was
indeed the case. This could be illustrated by a number of results. The rapid decline in bond began after the 250°C interface temperature was reached, Fig. 55. The results of the apparent slip-temperature relationship for the different covers tested show that only for temperatures up to approximately 300°C did the cover not affect the slip performance during the heating up period, Fig. 44. A tremendous increase in the irrecoverable slip occurred at 400°C compared with 250°C, see Fig. 69 for the load cycled tests. The steel stress distribution deteriorated between the 250 and 400°C interface temperature levels as Figs. 106(a), (b), and (c) indicate, which in turn produced an increase in the calculated anchorage length required after the 250°C value, see Fig. 117.

Hence a consistent pattern has been apparent throughout the test results showing that, between the 250 and 400°C interface temperatures, the bond performance deteriorated compared to that in the 100 to 250°C temperature range. This demonstrates agreement with the accepted practice of disregarding concrete heated to beyond 300°C.

7.6.1.2 Temperatures in the 100-300°C range

The reduction factor for the steel stress suggested in ref. 11 was 0.7. From the bond results it is possible to quantify this fire-damage factor. Fig. 83, shows that the bond was affected by heat to a greater extent than the concrete compressive strength as discussed in 5.3.1. This figure shows that the greatest reduction in bond up to 250°C was 82% of that at ambient conditions, giving a fire-damage factor of 0.82. This value was arrived at on the basis of the maximum bond stress-temperature relationship.

However as was stated in 2.1.4 it is not the maximum bond stress that is necessarily of importance, but rather, the bond stress in relation to the bond slip. This point is demonstrated by the 150 and 450°C nominal temperature results.
The maximum bond stresses were relatively close to each other (see Fig. 55) although one was suitable for reinstatement but the other was not. The difference was in the bond stress-slip relationship as seen in Figs. 38, 39 and 50-52. In view of this the value for the fire-damage factor of 0.82 was checked by an alternative method using the bond stress-slip curves, which in turn provided the steel stress distributions.

To do this Fig. 117 was replotted in Fig. 136 in terms of the decrease with temperature in the steel stress developed over an anchorage length consistent with that required to maintain an arbitrary steel stress at 20°C. Three steel stresses i.e., 100, 200 and 300 N/mm², were examined at ambient conditions to obtain the required anchorage length corresponding to an initial value of end slip equivalent to that occurring at the critical point under ambient conditions i.e. 20°C. Then the steel stress developed over each length for an increase in temperature was obtained from Figs. 112-116. Subsequently this was expressed as the ratio of steel stress at elevated temperature to steel stress at 20°C.

E.g. consider the point on Fig. 136 for an arbitrary steel stress of 300 N/mm² and a temperature level of 250°C. From Fig. 119, for an initial slip value equivalent to that at the critical value a distance of 152mm is needed to develop the steel stress required. Therefore from Fig. 113, for an initial slip corresponding to that at the critical value and a distance of 152mm, the steel stress developed is 246 N/mm².

\[
\text{residual steel stress at } 250°C = \frac{246}{300} = 0.82
\]

As Fig. 136 shows the decrease in the steel stress which could develop varied between 0.76 and 0.82 for temperatures up to 250°C.

However these values refer to the unstressed, residual condition, whereas for reinstatement purposes it is the
stressed, residual situation that is relevant. In this case the greatest reduction in bond (derived from Fig. 56) up to 250°C, was 88% of that at ambient conditions, as opposed to the 82% mentioned above. Therefore the procedure mapped out above and in Fig. 136 was repeated for test condition (c) (working stress, residual condition), and for each cover. The results are shown in Fig. 137, for an ambient steel stress of 300 N/mm². There are differences between the four different curves caused by experimental scatter, any effect of the cover magnitude, and the fact that the points for the smaller covers are derived from the results of four specimens of one batch as opposed to the five specimens each from a different batch in the case of the 55mm cover (section 5.1.3.3). However the general pattern is clear, that of a reduction in the bond effectiveness on initial heating followed by a further reduction after the 250°C temperature level. Therefore from the results it would appear that the reduction factor value of 0.7 suggested in ref. 11 gives a safe, satisfactory estimate of the steel stress due to the reduced anchorage caused by heating to temperatures between 100 and 300°C, for the type and grade of concrete utilised here. The points for the higher temperatures could not be obtained as the critical value was exceeded during the heating cycle.

7.6.2 Designing for Fire Resistance

When designing concrete members for a suitable fire resistance, it is vital to pay careful attention to the detailing of the reinforcement in order to ensure that adequate anchorage of the bar is provided.

This subject is dealt with elsewhere and a number of ways of allowing for a reduction in bond stress are discussed (20) e.g., bobs, hooks, bends and mechanical anchorage or extending the bar beyond the point where it is no longer required in any design condition. Detailing has to allow also for possible changes in the way a structural member behaves, for example, a simply supported beam, where in extreme cases a tied arch action can be set up provided sufficient
anchorage is present either by bond or mechanical means.

Therefore the fact that a reduction in bond stress has to be allowed for and provided against is well recognised. However little information is available on the actual extent of this reduction for various elevated temperatures. Using the bond test results obtained here it is possible again to quantify this reduction, which could help in estimating how much provision has to be made in the design process.

When designing a specified fire resistance into concrete members it is the stressed at elevated temperature condition which is of particular interest. As has been commented on (cf. section 5.1.3.1.2) for temperatures up to 250°C this condition showed a slightly greater reduction in maximum bond strength compared with the stressed, residual condition i.e., 84% as opposed to 88% as obtained from Fig. 56.

When considered in terms of the bond stress-slip relationship and the steel distribution curves, as carried out in 7.6.1.2 above, this difference made little impact on the results, see Fig. 138 (curves for test conditions (a) and (c)). The two unstressed conditions (b) and (d) are plotted also for completeness and give an indication of the curve shape at temperatures of 600 and 750°C. However it is likely that differences will be apparent at these higher temperatures for the two stressed conditions as reflected in the maximum bond stress results given in Fig. 55. From Fig. 138 the reduction in bond performance at temperatures above 250°C is again in evidence. From information of the type given in Figs. 137 and 138 it could be suggested that for fire resistant design, possible values are available for estimating the reduction in the bond performance, for anchorage, due to heating effects at various elevated temperature levels. The direct application of these figures is restricted to concrete mixes of the same pedigree as that used in the test programme, and for 16mm Tor bars.
If, when the fire resistance is computed, it is found that the reduction in bond prevents the required fire resistance being obtained, it would be possible to adjust the design to overcome this e.g., by extending the bar, or providing hooks etc.
8.1 CONCLUSIONS

The results obtained have been stated and discussed throughout the text. However the overall conclusions are now set down.

1. The tensile and compressive strength-temperature relationships showed differing characteristics, especially at temperatures around 250°C.

2. The residual stress-strain curve for mild and cold worked steels followed the established pattern i.e., mild steel had good recovery of strength on cooling while the cold worked bars began to revert to their original pre-cold work state.

3. A slight decrease in the rate of temperature rise produced a small decrease in the rate of slip.

4. A critical point in the bond stress-slip relationship was established which was caused by the crushing of the concrete immediately beneath the ribs. This point was exceeded either during the loading to failure or during the heating cycle when the heat had sufficiently reduced the concrete strength properties.

5. The critical point marked the beginning of the formation of a fracture surface at the top of the tooth of concrete between the ribs. The completion of this fracture surface occasionally produced another 'kink' in the $\sigma_b$-slip curve. This did not always take place due to the height of the tooth involved, the lower tooth portion preventing excessive slip from occurring.
6. Cover had little effect on the bond performance up to 250°C for bond stress values within the critical value range.

7. At maximum bond stress values specimens with different covers failed in different ways. The maximum bond stress-temperature relationship for the larger covers followed the concrete compressive strength-temperature curve with relatively large slips being observed. Hence a pull through type failure occurred with the concrete in compression in the vicinity of the ribs resisting this action. The smaller covers followed the tensile strength-temperature curve and gave very small slips, showing a tensile splitting failure mode to be predominant.

8. The bond performance is very much dependent on the concrete strength.

9. High steady-state bond stresses affected the results to a greater extent as the temperature increased, compared with low bond stress values.

10. Plain bars gave a less effective bond performance than deformed bars. However the overall shape of the maximum bond stress-temperature curves is consistent for both bar types.

11. As for concrete compressive strength, the stressed specimens gave a slightly better bond performance compared with the unstressed ones, when subjected to elevated temperatures.

12. Specimens tested in the residual condition gave more favourable results than those tested hot, for temperatures up to 250°C, due to the additional thermal stresses involved in the latter case. This pattern then
reversed as the temperature increased, coming into line with the characteristics observed for concrete compressive strength.

13. Load cycling reduced the maximum bond stress available. Also large irreversible slip began to take place during the cycling process between temperatures of 250-400°C.

14. The bond performance was reduced on initial heating, which was followed by a further deterioration for temperatures between 250 and 400°C. Thus coinciding with the 300°C value above which concrete is considered not suitable for reinstatement purposes.

15. The reduction in the bond strength was greater than the corresponding reduction in the concrete compressive strength. This was probably due to the increased thermal movement caused by the presence of the steel bar.

16. Acoustic emission is closely related to the bond slip indicating that it could be a useful method of investigating the quality of the bond in fire damaged structures.

17. By forming a second order differential equation and using the bond stress-slip results, slip, steel and bond stress distributions along a length of bar can be obtained. This enables certain beam situations to be examined.

18. The crack spacing for the unstressed residual test condition can be approximated to by utilising the steel stress distributions and taking account of the concrete strains.

19. The bond stress is so reduced that in the anchorage zone of a beam the allowable steel stress can be
considered to decrease by a factor of 0.7 for temperatures up to 250°C.

20. Quantifying the reduction in bond due to elevated temperatures enables a more accurate assessment of the quality of heat treated reinforced concrete members to be made. Such information can be useful when designing for reinstatement and fire resistant situations.

8.2 FUTURE WORK

In this work a number of different parameters affecting the bond have been investigated. However this amounts to only a small portion of the total still to be examined e.g., one type of concrete was studied along with one bar size, whereas results for a range for both of these parameters is required. Some of these parameters have been or are in the process of being looked at elsewhere e.g. Sager and Rostasney (47) have considered different concrete types, while Khan (appendix 6) has tested 8mm diameter bars. Work is also being conducted at Edinburgh into temperature effects on bond in no-fines concrete. However there still remain further variables to be considered. From the results in this work two variables in particular are apparent. These are rate of temperature rise and load cycling effects, both of which are pertinent to the case of fires in buildings. The load cycling variable is particularly relevant to the reinstatement of structures.

Another area which warrants further study is that of the acoustic emission application to fire damage assessment in general and bond in particular. This work has shown that A.E. can be correlated to bond slip. However further study is required to examine the effect of different parameters and also its application to individual members and subsequently to full-scale situations. Work is being conducted by M.R. Khan on beam tests which incorporate the monitoring of
the A.E. signals. Such beam tests (or tests on individual members) are the next step up from simple pull-out specimens used in this work. The pull-out test gives information on the bond, isolated from the other actions that take place, within for example a beam. Whereas from a study of the beam behaviour it would be possible to examine more the interaction between the various factors involved e.g., bond and shear. After tests on individual members have been carried out, to arrive at any meaningful practical application of A.E. to fire damage assessment, tests on full-scale structures would have to be undertaken.

One final area for future work pinpointed by this research project is the concrete stress-strain curve in tension for the hot and residual conditions. The author is unaware of any published work in this field. However as the calculations of the crack spacing set down in 7.4.2 showed, this information could be of use in obtaining accurate theoretical results, especially for cases of low steel stress levels and hence areas where the contribution of the concrete in resisting the forces is relatively high. Associated with this is the effect of temperature on the tensile strength of concrete, which would appear, from the tests carried out, to vary in a different way from that of the compressive strength-temperature relationship in that it progressively decreased with temperature.
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<thead>
<tr>
<th>No.</th>
<th>Author(s)</th>
<th>Title/Abstract</th>
</tr>
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<tbody>
<tr>
<td>38.</td>
<td>GHAHRAMANI A. &amp; SABZEVARI A.</td>
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<td>REICHEL V.</td>
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<table>
<thead>
<tr>
<th>CONCRETE COLOUR</th>
<th>TEMPERATURE (°C)</th>
<th>OTHER POSSIBLE PHYSICAL EFFECTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>40</td>
<td>None</td>
</tr>
<tr>
<td>Pink to Red</td>
<td>300</td>
<td>Surface crazing</td>
</tr>
<tr>
<td></td>
<td>550</td>
<td>Deep cracking</td>
</tr>
<tr>
<td></td>
<td>575</td>
<td>Popouts over chert or quartz aggregate particles.</td>
</tr>
<tr>
<td>Grey to Buff</td>
<td>600</td>
<td></td>
</tr>
<tr>
<td></td>
<td>800</td>
<td>Spalling, exposing not more than 25% of reinforcing bar surface.</td>
</tr>
<tr>
<td></td>
<td>900</td>
<td>Powdered, light coloured, dehydrated paste.</td>
</tr>
<tr>
<td>Buff</td>
<td>950</td>
<td></td>
</tr>
</tbody>
</table>

Table 1(13)

The physical effects of heat on concrete enabling the temperatures involved to be estimated.
<table>
<thead>
<tr>
<th>WORKER</th>
<th>HEATED OR RESIDUAL</th>
<th>SPECIMEN DIMENSIONS</th>
<th>BOND LENGTH</th>
<th>BAR TYPE</th>
<th>BAR DIA</th>
<th>CONCRETE STRENGTH AT 20°C</th>
<th>BOND STRENGTH AT 20°C</th>
<th>SPECIMEN AGE AT TEST</th>
<th>HEATING RATE</th>
<th>TEMP. RANGE</th>
<th>TIME FOR COOLING</th>
<th>DURATION AT MAX. TEMP.</th>
</tr>
</thead>
<tbody>
<tr>
<td>NILOVANOV(36) ERYADKO</td>
<td>Heated Res.</td>
<td>Prisms 140x140x300</td>
<td>300</td>
<td>Plain Def.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2 months</td>
<td>0.58 - 1.25</td>
<td>20-450</td>
<td>-</td>
</tr>
<tr>
<td>As reported by BUSHEV et al.(37)</td>
<td>Res.</td>
<td>-</td>
<td>-</td>
<td>Plain Def.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>20-450</td>
<td>-</td>
</tr>
<tr>
<td>GHANAMANI(38) SABZEVARI</td>
<td>Res.</td>
<td>Cubes 153x153x153</td>
<td>153</td>
<td>Plain Def.</td>
<td>-</td>
<td>21.4</td>
<td>-</td>
<td>-</td>
<td>28 days</td>
<td>slow (over 24 hrs)</td>
<td>20-150</td>
<td>-</td>
</tr>
<tr>
<td>HARADA et al.(39)</td>
<td>Res.</td>
<td>Cylinders 200x100 dim</td>
<td>200</td>
<td>-</td>
<td>-</td>
<td>42.8</td>
<td>-</td>
<td>-</td>
<td>26 months</td>
<td>0.5</td>
<td>20-450</td>
<td>48 hrs</td>
</tr>
<tr>
<td>KASAMI et al.(40)</td>
<td>Res.</td>
<td>Cylinders 200x100 dim</td>
<td>200</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>90 days</td>
<td>0.167</td>
<td>20-300</td>
<td>0.167°C/min</td>
</tr>
<tr>
<td>KNELEY(41)</td>
<td>Res.</td>
<td>Prisms 75x75x300</td>
<td>300</td>
<td>Square twist</td>
<td>12</td>
<td>38.6</td>
<td>-</td>
<td>-</td>
<td>28 days</td>
<td>4</td>
<td>20-820</td>
<td>24 hrs</td>
</tr>
<tr>
<td>REICHEL(42)</td>
<td>Res.</td>
<td>Prisms 150x150x450</td>
<td>300</td>
<td>Plain Def.</td>
<td>14</td>
<td>17.0</td>
<td>33.0</td>
<td>-</td>
<td>26 weeks</td>
<td>Standard fire test</td>
<td>20-685</td>
<td>24 hrs</td>
</tr>
<tr>
<td>HERTZ(43,44)</td>
<td>Res.</td>
<td>Conical see Fig. 13(f)</td>
<td>115</td>
<td>Plain Def.</td>
<td>16</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>28 days</td>
<td>1</td>
<td>20-800</td>
<td>7 days</td>
</tr>
<tr>
<td>DIEDERICH(45,46) SCHNEIDER</td>
<td>Heated</td>
<td>Cylinders 192x176 dim</td>
<td>80</td>
<td>Plain Def.</td>
<td>16</td>
<td>50.7-63.9</td>
<td>-</td>
<td>5.5</td>
<td>150-600 days</td>
<td>1</td>
<td>20-800</td>
<td>N/A</td>
</tr>
<tr>
<td>SAGER(47) POSTAST</td>
<td>Heated</td>
<td>Cylinders 192x176 dim</td>
<td>80</td>
<td>Def.</td>
<td>16</td>
<td>35.0</td>
<td>62.5</td>
<td>-</td>
<td>200 days</td>
<td>1</td>
<td>20-700</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Table 2
Summary of present work carried out on bond at elevated temperatures and in the residual condition.
<table>
<thead>
<tr>
<th>SERIES NO.</th>
<th>TEST CONDITIONS</th>
<th>COVER (mm)</th>
<th>BAR TYPE</th>
<th>STEADY STATE BOND STRESS (N/mm²)</th>
<th>LOAD CYCLING</th>
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<tr>
<td>AV 20-750</td>
<td>(a) (b) (c) (d)</td>
<td>55</td>
<td>deformed</td>
<td>3.70 0.00 3.70 0.00</td>
<td>No</td>
</tr>
<tr>
<td>AV 20-750</td>
<td>(a) (b) (c) (d)</td>
<td>55</td>
<td>deformed</td>
<td>3.70 0.00 3.70 0.00</td>
<td>No</td>
</tr>
<tr>
<td>AX 20-750</td>
<td>(a) (b) (c) (d)</td>
<td>55</td>
<td>deformed</td>
<td>3.70 0.00 3.70 0.00</td>
<td>No</td>
</tr>
<tr>
<td>AY 20-750</td>
<td>(a) (b) (c) (d)</td>
<td>55</td>
<td>deformed</td>
<td>3.70 0.00 3.70 0.00</td>
<td>No</td>
</tr>
<tr>
<td>AZ 20-750</td>
<td>(a) (b) (c) (d)</td>
<td>55</td>
<td>deformed</td>
<td>3.70 0.00 3.70 0.00</td>
<td>No</td>
</tr>
<tr>
<td>BX 20-750</td>
<td>(c)</td>
<td>46</td>
<td>deformed</td>
<td>3.70</td>
<td>No</td>
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<tr>
<td>BY 20-600</td>
<td>(c)</td>
<td>32</td>
<td>deformed</td>
<td>3.70</td>
<td>No</td>
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<tr>
<td>BZ 20-450</td>
<td>(c)</td>
<td>25</td>
<td>deformed</td>
<td>3.70</td>
<td>No</td>
</tr>
<tr>
<td>C 20-600</td>
<td>(c)</td>
<td>55</td>
<td>deformed</td>
<td>2.45</td>
<td>No</td>
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<tr>
<td>D 20-750</td>
<td>(c)</td>
<td>55</td>
<td>plain</td>
<td>2.45</td>
<td>No</td>
</tr>
<tr>
<td>E 20-750</td>
<td>(c)</td>
<td>55</td>
<td>deformed</td>
<td>3.70</td>
<td>Yes</td>
</tr>
<tr>
<td>F 20-750</td>
<td>(d)</td>
<td></td>
<td></td>
<td>Concrete strength specimens equivalent to the 55mm bond specimens.</td>
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</tbody>
</table>

* Furnace temperature levels between 20°C and an upper limit (individual levels of 20, 150, 300, 450, 600 and 750°C).

+ Different test conditions of the four specimens per batch.

++ All four specimens in a batch subjected to the same conditions.

Table 3 Outline of test programme
<table>
<thead>
<tr>
<th>COVER (mm)</th>
<th>NOMINAL FURNACE TEMPERATURE (°C)</th>
<th>BOND INTERFACE TEMPERATURE (°C)</th>
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<tr>
<td>55</td>
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Table 4

Temperature at the concrete/steel interface for the various covers and furnace temperatures.
<table>
<thead>
<tr>
<th>$T_f$ (°C)</th>
<th>$E_e$ (N/mm$^2$)</th>
<th>$m$</th>
<th>$\sigma_{ct}$ (N/mm$^2$)</th>
<th>$\sigma_{sxo}$ (N/mm$^2$)</th>
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<tr>
<td>20</td>
<td>14500</td>
<td>13.79</td>
<td>2.0</td>
<td>27.59</td>
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<td>150</td>
<td>11310</td>
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<td>300</td>
<td>7685</td>
<td>26.025</td>
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<td>40.34</td>
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<td>450</td>
<td>6960</td>
<td>28.74</td>
<td>1.14</td>
<td>32.76</td>
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<td>600</td>
<td>5510</td>
<td>36.30</td>
<td>0.73</td>
<td>26.50</td>
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<tr>
<td>750</td>
<td>3480</td>
<td>57.47</td>
<td>0.48</td>
<td>27.59</td>
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</tbody>
</table>

Table 5
Residual values of $E_e$, $m$, $\sigma_{ct}$ and $\sigma_{sxo}$ for various temperature levels.
A - Desorption and dehydration of tobermorite gel.
B - Dehydration of CaOH$_2$.O.
C - Dehydration of tobermorite gel.

Fig. 1(5) Linear shrinkage of cement paste with increasing temperature. (Heating rate: 5°C per minute.)

Fig. 2(7) Trends in unrestrained thermal expansion of concretes (diagrammatic; from work by Purkiss).
Fig. 3(7) Strains measured on concrete specimens heated and cooled under load (from work by Fisher); heating rate: 150°C per hour; B-stress maintained during heating/original strength; temperature maintained at 600°C for 3 hours.
Fig. 4(8) Temperature-strength relationship. Mix proportions, 1:6; water/cement ratio, 0.65.

Fig. 5(7) Effect of temperature on Young's modulus of concrete (from work by Philleo).
Fig. 6(7) Time-dependent strains in concrete maintained under load at high temperatures (from work by Cruz). Applied stress, 12.5 N/m².

Fig. 7(11) Yield strength of steels tested at elevated temperatures and in the residual condition.
(a) Tensile principal stresses at interface (After ref. 26) (b) Internal cracking in concrete at interface as observed (After ref. 27)

Fig. 9(24) Internal stresses and cracking in axially reinforced concrete prism subjected to tension

Fig. 9(24) Wedging action in resisting pull-out
Fig. 10 Methods of measuring steel strain.

Fig. 11 Method of measuring steel and concrete strain.
Fig. 12 Bond specimens with small embedment length.

(a) Rehm (33)
(b) Edwards and Yannopoulos (35)
Fig. 13 Different specimens used for bond tests at elevated temperatures.
Fig. 14 Residual concrete compressive strength-temperature relationship.
Fig. 15 Maximum residual bond stress-temperature relationship for plain bars.
Fig. 16 Maximum residual bond strength-temperature relationship for deformed bars.
Fig. 17 Maximum bond stress-temperature relationship for deformed bars with various aggregate types and concrete strengths.
Fig. 18 Specimen adopted for experimental investigation
Fig. 19 Details of method of applying load

Dimensions in mm
Fig. 20 Slip measuring arrangement
(a) Details of heating control

(b) Connection between elements in zone 1

Fig. 21 Furnace heating control system
Fig. 22 Details of furnace construction

Fig. 23 Details of furnace lid
Fig. 24 Details of test furnace
Fig. 25 Overall view of test furnace
Fig. 26 Details of casting table
(a) Position of thermocouples

(b) Radial temperature distribution

Fig. 27 Radial temperature distribution within a 55mm cover test specimen; radius, 63mm; heating rate, 2°C/min; time (t) in minutes.
Fig. 28 Vertical temperature distribution within a 55mm cover test specimen; radius, 63mm; heating rate, 2°C/min; time (t) in minutes.
Fig. 29 Residual concrete compressive strength-mean temperature relationship.

Fig. 30 Residual concrete tensile strength-mean temperature relationship. From 55mm cover, plain and deformed bar specimens.
Fig. 31 Relative "hot" strengths of concrete. "Cold" strength at room temperature 100 percent. (Wierig 1965).
Fig. 32 Stress-strain curves for reinforcing steel heated to various temperatures and tested on cooling.
Fig. 33 Nominal furnace and interface temperature distribution curves on heating to the temperatures examined. Cover, 55mm.
--- Movement during the one hour period at a constant temperature level.

Fig. 34 Movement-temperature curves during the heating up period. 16mm deformed bar; cover, 55mm; steady state bond stress, 0.00 and 3.70 N/mm²; (T₁) in °C.
Fig. 35 Apparent slip-temperature relationship during the heating up period. 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm²; \( T_r \) in °C.
Fig. 36 Movement-time curves for various temperatures during the 24 hour cooling period. 16mm deformed bar; cover, 55mm; $T_i (T_f)$ in °C.
Fig. 37 Apparent slip-time relationship for various temperatures during the 24 hour cooling period. 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm²; \( T_f \) in °C.
Fig. 38 Residual bond stress-slip curves for various temperatures. Test condition (c): stressed, residual; 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm²; T₁ (T₁) in °C.

Fig. 39 Enlargement of initial part of residual bond stress-slip curves shown in Fig. 38.
crushing beneath rib

fracture surface beginning at critical value

final fracture surface

\( \sigma_{cn} \) = normal stresses in concrete under rib.

\( \sigma_{bp} \) = bond stress due to sliding resistance of smooth part of bar.

\( \sigma_{os} \) = shear stresses in concrete.

**Fig. 40** Stress components and fracture surfaces in the region of the concrete/steel interface.
Fig. 41 Variation of concrete compressive strength with temperature. 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm².
Fig. 42 Range of the completion of the fracture surface for various temperatures.

Fig. 43 Maximum residual bond stress-temperature relationship for the stressed, residual condition. 16mm deformed bar; cover 55mm; steady state bond stress, 3.70 N/mm².
Fig. 44  Apparent slip-temperature relationship during the heating up period for various cover sizes. 16mm deformed bar; steady state bond stress, 3.70 N/mm$^2$. 

--- 25mm cover
--- 32mm cover
--- 46mm cover
--- 55mm cover
Fig. 45 Apparent slip-temperature relationship during the heating up period for different magnitudes of steady state bond stress. 16mm deformed bar; cover, 55mm.
Fig. 46 Apparent slip-temperature relationship during the heating up period for plain and deformed bars. Cover, 55mm.

- a - 16mm plain bar; 2.45 N/mm² steady state
- b - 16mm deformed bar; 3.70 N/mm² bond stress
- c - 16mm deformed bar; 2.45 N/mm²

+ point where $\sigma_{cn}$ is exceeded
Fig. 47  Movement-time curves during the 24 hour cooling period for various temperatures and covers. 16mm deformed bar; steady state bond stress, 2.70 kN/mm²; (Tₐ) in °C.
Fig. 48 Apparent slip-time relationship during the 24 hour cooling period for various temperatures. 16mm deformed bar; cover, 55mm; steady state bond stress, 2.45 N/mm²; ($T_f$) in °C.

Fig. 49 Apparent slip-time relationship during the 24 hour cooling period for various temperatures. 16mm plain bar; cover, 55mm; steady state bond stress, 3.70 N/mm²; ($T_f$) in °C.
Fig. 50 Bond stress-slip curves for various temperatures. Test condition (a): stressed, hot; 16 mm deformed bar; cover, 55 mm; steady state bond stress, 3.70 N/mm²; $T_i$ ($T_f$) in °C.
Fig. 51 Bond stress-slip curves for various temperatures. Test condition (b): unstressed, hot; 16mm deformed bar; cover, 55mm; $T_i$ ($T_f$) in °C.
Fig. 52 Residual bond stress-slip curves for various temperatures. Test condition (d): unstressed, residual; 16mm deformed bar; cover, 55mm; $T_i$ ($T_f$) in °C.
Fig. 53 Maximum bond stress-temperature relationship for hot test conditions (a) and (b). 16mm deformed bar; cover, 55mm; steady state bond stress, 0.00 and 3.70 kN/mm².
Fig. 54 Maximum bond stress-temperature relationship for residual test conditions (c) and (d). 16mm deformed bar; cover, 55mm; steady state bond stress, 0.00 and 3.70 N/mm².
Fig. 55 Maximum bond stress-temperature relationship for test conditions (a)-(d). 16mm deformed bar; cover, 55mm; steady state bond stress, 0.00 and 3.70 N/mm².
Fig. 56 Maximum bond stress-temperature relationship for stressed test conditions (a) and (c). 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm².
Fig. 57 Maximum bond stress-temperature relationship for unstressed test conditions (b) and (d). 16mm deformed bar; cover, 55mm.
Fig. 58 Variation of concrete compressive strength with temperature for various test conditions. 16mm deformed bar; cover, 55mm; steady state bond stress, 0.00 and 3.70 N/mm².
Figs. 59-61 Residual bond stress-slip for different covers at various temperatures. 16mm deformed bar; steady state bond stress, 3.70 N/mm²; $T_i$ ($T_f$) in °C.
Fig. 62 Residual maximum bond stress-temperature for various covers. 16mm deformed bar; steady state bond stress, 3.70 N/mm².
Fig. 63 Variation of concrete compressive strength with temperature for various covers. 16mm deformed bar; steady state bond stress, 0.00 and 3.70 N/mm².
Fig. 64 Residual bond stress-slip curves for various temperatures. 16mm deformed bar; cover, 55mm; steady state bond stress, 2.45 N/mm²; $T_1$ ($T_f$) in °C.
Fig. 65 Maximum residual bond stress-temperature relationship for test series C. 16mm deformed bar; cover, 55mm; steady state bond stress, 2.45 kN/mm².
Fig. 66 Variation of concrete compressive strength with temperature for test series C. 16mm deformed bar; cover, 55mm; steady state bond stress, 2.45 N/mm².
Fig. 67 Residual bond stress-slip for 16mm plain bars at various temperatures. Cover, 55mm; steady state bond stress, 2.45 N/mm²; $T_i (T_f)$ in °C.

--- specimens failed under steady state bond stress before the temperature level was reached

Fig. 68 Maximum residual bond stress-temperature relationship for 16mm plain bars. Cover, 55mm; steady state bond stress, 2.45 N/mm².
Fig. 69 Residual stress-slip during load cycling for various temperatures. 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm$^2$; load cycled between 1.0 and 3.70 N/mm$^2$; $T_i$ ($T_f$) in °C.
Fig. 70 Residual bond stress-slip during load cycling for various temperatures. 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm²; load cycled between 1.0 and 3.70 N/mm²; \( T_i (T_f) \) in °C.
Fig. 71 The effect of load cycling on the maximum bond stress-temperature relationship. 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm².
(a) Predominant crack pattern

(b) Advanced crack formation

Fig. 72 General crack pattern
Fig. 73 Predominant crack pattern. 16mm deformed bar; cover, 25mm; bond stress during heating, 3.70 N/mm²; temperature notation $T_i (T_f)$ in °C, 20 (20).
Fig. 74 Normal pattern of little cracking at 20°C for higher covers with one exceptional case. 16mm deformed bar; cover, 46mm; bond stress during heating, 3.70 N/mm²; temperature notation $T_i$ ($T_f$) in °C; 20 (20).
Fig. 75 Predominant crack pattern. 16mm deformed bar; cover, 46mm; bond stress during heating, 3.70 N/mm²; temperature notation $T_i (T_f)$ in °C, 100 (150).

Fig. 76 Predominant crack pattern. 16mm deformed bar; cover, 32mm; bond stress during heating, 3.70 N/mm²; temperature notation $T_i (T_f)$ in °C, 250 (300).
Fig. 77 Predominant crack pattern. 16mm deformed bar; cover, 32mm; bond stress during heating, 3.70 N/mm²; temperature notation $T_1$ ($T_f$) in °C, 100 (150).
Fig. 78 Advanced crack formation. 16mm deformed bar; cover, 46mm; bond stress during heating, 3.70 N/mm²; temperature notation $T_t$ ($T_f$) in °C, 565 (600).
(a) Initiation of transverse cracks.

(b) A deflected specimen form at an advanced stage of cracking.

Fig. 79 Crack initiation
Fig. 80 Little cracking due to load but hairline cracks caused by heating. 16mm deformed bar; cover, 55mm; bond stress during heating, 3.70 N/mm²; temperature notation $T_i (T_f)$ in °C, 720 (750).

Fig. 81 Radial cracking that has not reached the concrete surface.
(a) Mortar attached to steel bar after removal from the test specimen

(b) Slip of bar within a concrete specimen.

Fig. 82 Steel bar and concrete specimens after the splitting test.
Fig. 83 Comparison between concrete and bond strength for the unstressed, residual condition.
Fig. 84 Method of obtaining the acoustic emission signals

Fig. 85 Outline of acoustic emission instrumentation
Fig. 86 A.E. counts-temperature and apparent slip-temperature relationship during the heating up period for 450 and 600°C. 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm²; $T_i (T_f)$ in °C.
Fig. 87 A.E. counts-time relationship during the first 30 minutes of cooling. 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm²; $T_1 (T_2)$ in °C, 565 (600)
Fig. 88 Development of A.E. with residual bond stress and slip for ambient conditions. $T_i$ $(T_r)$, 20 (20); 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm².

At maximum $\sigma_b$ number of A.E. counts = 23370
Fig. 89 Development of A.E. with residual bond stress and slip. $T_i (T_f)$, 100 (150); 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm².

At maximum $\sigma_b$ number of A.E. counts = 29790
At maximum $\sigma_b$ number of A.E. counts = 42350

Fig. 90 Development of A.E. with residual bond stress and slip. $T_i$ ($T_f$), 250 (300); 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm².
At maximum $\sigma_b$ number of A.E. counts = 21040

A - position of photograph in Fig. 99(a)
B - position of photograph in Fig. 99(b)

Fig. 91 Development of A.E. with residual bond stress and slip. $T_f$ ($T_r$), 400 (450); 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm$^2$. 
At maximum $\sigma_b$ number of A.E. counts = 649620

Fig. 92 Development of A.E. with residual bond stress and slip. $T_i$ ($T_f$), 565 (600); 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm².

At maximum $\sigma_{bp}$ number of A.E. counts = 1570

Fig. 93 Development of A.E. with residual bond stress and slip. $T_i$ ($T_f$), 20 (20); 16mm plain bar; cover, 55mm; steady state bond stress, 2.45 N/mm².
Fig. 94 Development of A.E. with residual bond stress and slip. $T_i (T_f)$, 100 (150); 16mm plain bar; cover, 55mm; steady state bond stress, 2.45 N/mm².

At maximum $\sigma_{bp}$ number of A.E. counts = 1170

Fig. 95 Development of A.E. with residual bond stress and slip. $T_i (T_f)$, 250 (300); 16mm plain bar; cover, 55mm; steady state bond stress, 2.45 N/mm².

At maximum $\sigma_{bp}$ number of A.E. counts = 2900
AER (%) 

![Graph showing AER (%) vs Slip (mm)]

- residual $\sigma_{bp}$-slip
- residual $\sigma_{bp}$-AER
- AER-slip

At maximum $\sigma_{bp}$ number of A.E. counts = 5890

A - position of photograph in Fig. 100

Fig. 96 Development of A.E. with residual bond stress and slip. $T_i (T_f)$, 400 (450); 16mm plain bar; cover, 55mm; steady state bond stress, 2.45 N/mm².
Fig. 97 Bond stress-A.E. counts for load cycled test at ambient condition.

Fig. 98 Bond stress-A.E. counts for load cycled test. $T_1 (T_p)$, 400 (450); 16mm deformed bar; cover, 55mm; steady state bond stress, 3.70 N/mm².
Fig. 99 A.E. waveform for a 16mm deformed bar bond test. $T_i (T_f)$ in °C - 400 (450).
A.E. waveform corresponding to A on Fig. 96 scale - 0.1 volts per cm.

Fig. 100 A.E. waveform for a 16mm plain bar bond test.
Fig. 101 Distribution of the A.E. counts over the height of a specimen. Ti (Tr), 20 (20); 16mm deformed bar; cover, 55mm; steady state bond stress, 0.00 N/mm².
Fig. 102 Slip as the difference between the steel and concrete strain.
Fig. 103 Breakdown of bond stress-slip curve into three portions.

C.V. - critical value

Fig. 104 Steel and concrete stress distributions between two cracks.
Fig. 105 Slip distributions for test conditions (a)-(d) and various temperatures. Datum initial conditions; 16mm deformed bar; cover, 55mm.
Fig. 106 Steel stress distributions for test conditions (a)-(d) and various temperatures. Datum initial conditions; 16mm deformed bar; cover, 55mm.
Fig. 107 Bond stress distributions for test conditions (a)-(d) and various temperatures. Datum initial conditions; 16mm deformed bar; cover, 55mm.
Fig. 108 Residual slip distributions for various covers and temperatures. Datum initial conditions; 16mm deformed bar; steady state bond stress, 3.70 N/mm².

Initial values: \( \Delta_o = 0.0001 \text{mm}; \sigma_{sx_0} = f(\Delta_o) \)
Fig. 109 Residual steel stress distributions for various covers and temperatures. Datum initial conditions; 16mm deformed bar; steady state bond stress, 3.70 N/mm².

Initial values: $\Delta_o = 0.0001$ mm; $\sigma_{sx_o} = f(\Delta_o)$
Fig. 110 Residual bond stress distributions for various covers and temperatures. Datum initial conditions; 16mm deformed bar; steady state bond stress, 3.70 N/mm².
Fig. 111 Distribution curves in the anchorage zone.
Test condition (d); unstressed, residual; 16mm deformed bar; cover, 55mm
Fig. 112 $T_i (T_f)$ in $^\circ$C - 100 (150)
C.V. - slip at critical value, 0.0084 mm

Initial values: $\Delta_0$ varies (a - 0.001, b - 0.002, c - 0.004, d - 0.006, e - 0.016,
f - 0.032, g - 0.064, h - 0.128 and i - 0.256 mm; $\sigma_{sx_0} = f(\Delta_0)$)

Figs. 112 and 113 Steel stress distribution curves in the anchorage zone.
Test condition (d): unstressed, residual; 16 mm deformed bar; cover, 55 mm

Fig. 113 $T_i (T_f)$ in $^\circ$C - 250 (300)
C.V. - slip at critical value, 0.0207 mm
Steel stress distribution curves in the anchorage zone.
Test condition (d); unstressed, residual; 16mm deformed bar; cover, 55mm.
Fig. 115 $T_i (T_f)$ in $^\circ C - 565 (600)$
C.V. - slip at critical value, 0.07 mm

Figs. 115 and 116 Initial values:
(a - 0.001, b - 0.002, c - 0.004,
d - 0.006, e - 0.016, f - 0.032,
g - 0.064, h - 0.128,
i - 0.256 mm); $\sigma_{x_0} = f(x_0)$

Fig. 116 $T_i (T_f)$ in $^\circ C - 620 (750)$
C.V. - slip at critical value, 0.1 mm

Test condition (d), unstrained, residual 16mm deformed bar; cover, 55mm
values are for end slips equivalent to the bond slip at the critical value for each temperature level respectively.

Fig. 117 - Effect of temperature on the anchorage length.
Test condition (d): unstressed, residual; 16mm deformed bar; cover, 55mm.
Fig. 118 Distribution curves in the cracking zone.
Test condition (d): unstressed; residual; 16mm deformed bar; cover, 55mm.
Fig. 119 $T_i (T_f)$ in °C - 100 (150)

Initial values: $\Delta_o = 0.0001$mm; $\sigma_{sx}^-$ varies (a = 1, b = 2, c = 4, d = 8, e = 16, f = 32, g = 64, h = 128, i = 256, and j = 350 N/mm²)

Figs. 119 and 120 Steel stress distribution curves in the cracking zone.
Test condition (d): unstressed, residual; 16mm deformed bar; cover, 55mm
Fig. 121 Steel stress distribution curves in the cracking zone.
Test condition (d); unstressed, residual; 16mm deformed bar; cover, 55mm
Referring to section 7.5.2

\[ \sigma_{sx_0} = 26.5 \text{ (see Table 5)} \]

\[ (\sigma_{sx_1} - \sigma_{sx_0}) = 56.94 \text{ (cf. 7.4.2.4)} \]

For constant bending conditions an intermediate crack is assumed to occur midway between the first two cracked sections i.e., at a distance of 180mm from the first crack. (see Fig. 124(b)).

Fig. 122 shows this occurring at an initial steel stress of about 130 N/mm².

No further cracking occurs as the necessary steel stress difference of 56.94 N/mm² is not developed over a distance of 180/2 = 90mm.

Minimum crack spacing for 600°C, for conditions consistent with those assumed for lower temperature values given in Figs. 132-134 = 180mm.

\[ T_i (T_r) \text{ in } °C - 565 (600) \]

Initial values: \( \Delta_o = 0.0001 \text{mm} \); \( \sigma_{sx_0} \) varies (a - 1, b - 2, c - 4, d - 8, e - 16, f - 32, g - 64, h - 128, i - 256, and j - 450 N/mm²).

Fig. 122 Steel stress distribution curves in the cracking zone.
Test condition (d); unstressed, residual; 16mm deformed bar; cover, 55mm
Fig. 123 Steel stress distribution curves in the cracking zone.

Test condition (d); unstressed, residual; 16mm deformed bar; cover; 55mm
**Fig. 124 Development of tensile cracking**

(a) Varying bending moment field

- prior to first crack
- effect of first crack
- prior to second crack
- effect of second crack
- prior to third crack
- $\sigma_t$
(b) Constant bending moment field

Fig. 124 Development of tensile cracking
tension compression

stress distribution over section zz

Fig. 125 Tensile stress distribution over a cylindrical concrete specimen during the tensile splitting test.

cover

(2 x cover) + bar diameter

Fig. 126 Area of concrete assumed to contribute to the tensile resistance in a beam.
Fig. 127 Crack spacing-temperature relationship
Fig. 128 $T_i (T_f)$ in $^\circ$C - 20 (20)

Initial values: $\Delta_0 = 0.0001 \text{mm}$; $\sigma_{sx0}$ varies (a - m. $\sigma_{ct} = 27.59$, b - 55.18, c - 110.36, d - 220.72, e - 300.0 N/mm$^2$)

Fig. 129 $T_i (T_f)$ in $^\circ$C - 100 (150)

Initial values: $\Delta_0 = 0.0001 \text{mm}$; $\sigma_{sx0}$ varies (a - m. $\sigma_{ct} = 32.54$, b - 65.08, c - 130.16, d - 260.32, e - 300.0 N/mm$^2$)

Figs. 128 and 129 Steel stress distribution curves in the cracking zone allowing for the concrete strain. Test condition (d): unstressed, residual; 16mm deformed bar; cover, 55mm.
Fig. 130  $T_1 (T_f)$ in °C - 250 (300)

Initial values: $\Delta_0 = 0.0001\text{mm}$; $\sigma_{sx_0}$ varies (a - m. $\sigma_{ct}=40.34$,
b - 80.68, c - 161.36, d - 322.72,
e - 350.0 N/mm²)

Figs. 130 and 131  Steel stress distribution curves in the cracking zone allowing for the concrete strain.
Test condition (d): unstressed, residual; 16mm deformed bar; cover, 55mm.
Fig. 132 Comparison between actual and calculated results for 40mm beams.
Fig. 133 Comparison between actual and calculated results for 30mm beams.

Fig. 134 Comparison between actual and calculated results for 20mm beams.
inclined cracks due to bond imposed shear and conventionally called shear action

(a) Failure pattern for temperatures between 20 and 500°C

crack extended to the bottom of the beam indicating that bond slip took place
diagonal splitting cracks due to ultimate work at free body and arch model

(b) Failure pattern for temperatures of 600°C and above

Fig. 135 Failure pattern of beams subjected to elevated temperatures (after M.R. Khan and ref. 20).
Fig. 136 Variation with temperature of the residual steel stress to the steel stress at 20°C. Test condition (d): unstressed, residual; 16 mm deformed bar; cover, 55 mm.
Fig. 137 Variation with temperature of the residual steel stress to the steel stress at 20\(^\circ\)C for different covers. Test condition (c): stressed, residual; 16mm deformed bar; steady state bond stress, 3.70 N/mm\(^2\); ambient steel stress, 300 N/mm\(^2\).
Fig. 138 Variation with temperature of the residual steel stress to the steel stress at 20°C for test conditions (a)-(d). 16mm deformed bar; cover, 55mm; steady state bond stresses, 0.00 and 3.70 N/mm²; ambient steel stress, 300 N/mm².
APPENDIX 1

The Influence Of High Temperature On The Bond In Reinforced-Concrete

by

P.D. Morley and R. Royles

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The Influence of High Temperature on the Bond in Reinforced Concrete

P. D. MORLEY and R. ROYLES
Department of Civil Engineering & Building Science, University of Edinburgh, Edinburgh EH9 3JL (Gt. Britain)
(Received November 23, 1979)

SUMMARY

A knowledge of the effects of temperature on the bond strength between steel and concrete in reinforced concrete is important if a complete understanding of its fire resistance and residual capacity for structural performance is to be known.

A summary of the strength of concrete and steel during, and after, exposure to elevated temperatures is made. This is followed by a review of the different procedures used for testing bond at ambient temperature. Work carried out on bond at high temperatures is discussed and suggestions made as to its relevance to structural performance. Preparations by the authors to investigate this problem more fully with the intention of providing information which will be of use in the design of fire resistance and the reinstatement of fire damaged reinforced concrete components are described.

INTRODUCTION

Fire is one of the major hazards to which a building can be subjected since not only is the safety of the occupant put at risk, but damage can be immense even to the extent of putting a firm out of business and rendering families homeless.

In order to help people escape from a fire a predetermined fire resistance has to be met which ensures the stability and integrity of a structure, e.g., a 2 hour fire resistance means that the building or member concerned can withstand a fire with the equivalent ferocity of 2 hours exposure to the standard test fire [1, 22]. However, if a structure is constructed in concrete it often remains standing after the fire has finished. This can lead to a situation in which it is often quicker and cheaper to reinstate the damaged parts of a structure as opposed to demolishing and reconstructing the whole of it.

Before reinstatement can begin it must be established that the damaged structure is suitable for such a treatment. In order to do this an assessment of its residual capacity for structural performance must be made and for such a task the effect of temperature on the properties of reinforced concrete must be known.

The main concern from the structural performance point of view is the condition of the basic materials, concrete and steel, and the bond between them. Information on the properties of steel both at high temperatures and after heat treatment is quite extensive. Similarly, a certain amount of knowledge exists regarding the effect of temperature on concrete, but little is known about the response to heat treatment of the bond between the two materials. Sufficient bond strength is essential if a member is to give a satisfactory structural performance, including gradual failure with adequate warning, rather than sudden collapse.

In this paper the behaviour of concrete and steel at high temperatures and after heat treatment is summarised as a prologue to an appraisal of the quality of the bond between them. Methods of establishing this quality both at normal and elevated temperatures are described and some plans for future investigations are outlined.

MATERIAL RESPONSE TO ELEVATED TEMPERATURE

Before studying the effect of temperature on the bond between steel and concrete it is important to know the effect temperature has on the two materials themselves.
(i) Concrete

As concrete is heated, changes take place in both the hardened cement paste and the aggregate. Within the cement paste there is both free water and water combined with hydration products. As the temperature increases, first the free water and then the chemically combined water is lost. This dehydration process causes the cement paste to shrink more than the expansion due to temperature rise.

At the same time the aggregate responds to the heat by expanding. The type of aggregate used affects the performance of the concrete due to the differing thermal properties. For example, a low thermal conductivity will slow up heat transfer from the hot surface to the cool interior. Also, a large coefficient of thermal expansion will cause greater movement and cracking problems compared with a smaller one.

The net result of the cement paste shrinking and the aggregate expanding is that stresses are set up within the concrete which eventually cause cracking of the cement paste and, hence, a loss of strength.

It has been noted that the type of aggregate affects the concrete response to elevated temperature (e.g., lightweight aggregate gives a better performance than siliceous aggregate) and this is also true for the different conditions under which concrete is tested. Investigators [2, 3] have reported that concrete under stress whilst subjected to heating gives a better performance than that under no stress conditions. Likewise, strength at high temperatures is greater than that after cooling has taken place.

For the practical application of assessing the strength of fire damaged concrete members the significant temperature is considered to be about 300 °C [4]. If the concrete is heated to below this level it is considered capable of being re-used, allowing for a slight loss in strength. It is suggested [4] that for redesigning purposes a strength of 0.85 times the original strength be assumed for concrete heated to between 100 and 300 °C.

The basic relationship between concrete strength and temperature is shown in Fig. 1 [5].

(ii) Steel

For reinforcing steel the loss of yield strength is significant while the steel remains hot but, except for cold worked steel, the recovery is practically complete after cooling from temperatures up to 700 °C [4]. The yield strength of cold worked steel does not recover so well since the heating process releases sessile dislocations created initially by the amount of prior cold working of the crystal structure. The relocking of dislocations on cooling occurs to a much lesser extent and, hence, the number of barriers to slip are reduced compared with the initial condition.

Figure 2 shows the basic curves for steel strength, hot rolled and cold worked, against temperature. The yield strength of steel is reduced by half at approximately 550 °C [6].

From this it can be seen that the critical time for the steel is during its period at the elevated temperature. The loss of steel strength
cause excessive deflections in flexural members and distortions in columns or, where the concrete cover has spalled away in compression zones, local buckling of a bar. In this respect spalling can lead to higher steel temperatures and lower strength and any buckling of steel bars in compression zones can compound spalling.

Special care must be taken when dealing with prestressing steel due to the loss of tension caused by the relaxation effects when creep occurs. In this case 50% of normal yield strength is likely to be achieved at 400 °C [6].

**BOND AT AMBIENT TEMPERATURE**

When dealing with bond quality it is not just the maximum bond strength available that is important but the bond stress in relation to the amount of slip between the steel and concrete. The most useful characteristic is the local bond resistance to slip since from this relationship attempts can be made to establish crack widths and spacing in loaded reinforced concrete members.

Over the years many different experiments have been undertaken mainly along the lines of either pull-out or beam tests [7]. Originally the average bond stress was obtained for a specific length of bar in accord with the following definitions.

For pull-out tests:

\[
\text{average bond stress} = \frac{\text{force in bar}}{\text{embedment length} \times \text{bar perimeter}}
\]

For beam tests:

\[
\text{average bond stress} = \frac{\text{change in tensile force in steel over a prescribed length}}{\text{bar perimeter} \times \text{prescribed length}}
\]

where the prescribed length could be the length from the end of the beam to a crack initiator or the length of the shear arm.

These definitions enabled different bar types to be compared and the anchorage zone behaviour to be investigated. However, the bond stress over a length of bar is not constant and this has led to tests being carried out to establish bond stress distribution. This has been achieved by measuring the steel strain and, hence, the steel stress at various points along a bar. A number of methods have been employed, e.g., holes have been provided in the concrete giving access to the steel bar, thus enabling strain to be measured [8]. Electrical resistance strain gauges have been attached to the outside of a bar and also to the inside of steel reinforcement; the bar being cut longitudinally, a groove milled out of one part of it, the gauges fixed and the pieces tack welded together again [9] (see Fig. 3).

All these methods have disadvantages. The access holes and strain gauges attached to the outside of the bar affect the amount of concrete being bonded to the steel. Even with gauges fixed to the bar internally, actual conditions will not be reproduced, the reduced cross-sectional area increasing the strain in the steel for a particular load.

Internal slip is even more difficult to measure due to the need to consider the concrete strain. It has been done utilising electrical resistance strain gauges to develop a slip gauge. One end of the slip gauge was bonded to the steel and the other end to a mortar block with the central portion unbonded. The mortar block was embedded in the concrete so that the gauge was parallel to the bar axis. Another approach was to measure the steel and concrete strain independently using electrical resistance strain gauges — some bonded to the steel and some to the concrete. By comparing the two, slip was calculated. It can be appreciated that the preparation of such specimens must be done with care and to obtain a continuous distribution along a bar length a great number of gauges are required [10] (see Fig. 4).

Rehm [11] approached the problem in a different way. Since it is the local bond-slip properties that are required to be tested a small ‘differential length’, e.g., a bond length of 16 mm, was employed. By doing this the stress-slip curve could be arrived at directly.

---

**Fig. 3. Methods of measuring steel strain.**
from the load applied and the measured end slip. This simplifies the experimental procedure considerably although great care must be taken in producing the specimens because of the small bond length. Edwards and Yannopoulos [12] used a similar procedure, increasing the bond length to 38 mm for practical reasons such as, for example, 19 mm aggregate was used and so for a specimen of this type it would be impractical to use a bond length of less than 19 mm. The Rehm and Edwards/ Yannopoulos specimens are shown diagrammatically in Fig. 5(a) and (b), respectively. These specimens can also be criticised for not being true to conditions in reality. The type in Fig. 5(a) gives greater lateral restraint than normal whilst those in Fig. 5(b) could be said to represent conditions at the beam end or crack face only.

The variability between the test results obtained in the different investigations might be explained, in part, by a lack of uniformity in the treatment of the following factors:


(ii) Surface condition of reinforcement. Smooth, clean steel surfaces and those with excessive rusting or scaling do not provide as good a basis for bonding with cement paste as a lightly corroded surface, see Rehm [11] (Section 3.22).

(iii) Relationship between (a) aggregate and bar size, (b) bar size and cover. The tensile strength of the concrete surrounding a bar is enhanced as the amount of concrete cover increases because a larger amount of concrete can be compacted more readily. Similarly, the smaller the aggregate size in relation to the bar diameter the more workable will be the concrete and the greater the chances of obtaining good adhesion between the cement paste and the steel.

(iv) Curing conditions. The curing conditions at the time of casting and the prevailing moisture content at the time of testing influence the shrinkage of the concrete and the concrete hoop stress surrounding a bar.

Whatever specimen is used, criticism can be levelled at it in some form. However, using the ‘differential bond length’ approach is cheaper and easier for testing and does give the required results directly. These points are even more pronounced when dealing with bond at high temperatures.

BOND AT ELEVATED TEMPERATURES

The present knowledge of the effect of temperature on the bond strength of rein-
forced concrete is very limited. However, a few tests have been carried out by different investigators and are considered below.

The works fall into two categories namely those concerned with the residual bond strength (i.e., after cooling) and those dealing with bond strength whilst at the elevated temperature.

(a) Residual bond strength

(1) In Japan work was undertaken by Harada et al. [13]. Cylinders 200 mm in length and 100 mm dia. (Fig. 6(a)) were heated at a rate of 0.5 °C/min to temperatures of 100, 300 and 450 °C, held there for 72 h and subsequently cooled slowly over the next 48 h. Then the residual bond strength was tested, and compared with the concrete strength of identical specimens subjected to the same heating cycle. Age and concrete strength at the time of testing was 26 months and 42.75 N/mm², respectively.

Readings of the maximum bond strength and bond at a slip of 0.05 mm were recorded. These are shown in Table 1 along with the compressive strength results.

TABLE 1
Variation of concrete compressive and residual concrete-steel bond strength with temperature according to Harada et al. [13].

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>20</th>
<th>100 - 300</th>
<th>450</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bond at 0.05 mm slip</td>
<td>100</td>
<td>44</td>
<td>10</td>
</tr>
<tr>
<td>Max bond strength (percentage of the 20 °C value)</td>
<td>100</td>
<td>50-60</td>
<td>10</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>100</td>
<td>75</td>
<td>60</td>
</tr>
</tbody>
</table>

The conclusion reached from this work was that when subjected to high temperatures the percentage reduction in bond strength is greater than that of concrete strength.

(2) Work done in Czechoslovakia by Reichel [14] showed differences in experimental procedure. Prisms of dimensions 150 × 150 × 450 mm length were used together with a 14 mm dia. bar embedded in the prism for a length of 300 mm (Fig. 6(b)). Supplementary steel reinforcement was required to prevent premature break-up of the concrete prism.

Fig. 6. Different specimens used for bond tests at elevated temperatures (a) Harada et al. [13], (b) Reichel [14], (c) Morley [15], (d) Ghahramani and Sabzevari [16], (e) Milovanov and Pryadko [17], (f) Schneider and Diederichs [20].
These test specimens were subjected to the standard fire test (i.e., temperature following the curve \( T = T_0 + 345 \log (8t + 1) \) where \( T_0 = \) initial temperature and \( t = \) time) with 24 h allowed for cooling. The temperatures reached were in the ranges 485 - 500 °C and 615 - 685 °C. The age of specimens at testing was approximately 26 weeks. In this work two concrete strengths, and two steel bars, deformed and plain, were used.

The results are given in Table 2.

**TABLE 2**  
Variation of residual concrete-steel bond strength with temperature according to Reichel [14]

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>20</th>
<th>500</th>
<th>650</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Bond Strength</td>
<td>Deformed Bar</td>
<td>100</td>
<td>75</td>
</tr>
<tr>
<td>Plain Bar</td>
<td>100</td>
<td>40</td>
<td>25</td>
</tr>
</tbody>
</table>

Reichel concluded that the decrease in bond is not greatly influenced by concrete quality, and also that the strength of steel does not undergo any great reduction after the heating cycle is completed, but as the bond strength does decrease, it can be considered a critical parameter.

(3) Some work has been carried out in Edinburgh [15] in which there were again differences in experimental procedure. The test specimens were prisms 75 X 75 X 300 mm long. Square, twisted bar (12 mm in size) was used embedded along the complete length of the prism (Fig. 6(c)). The effects of eight temperatures between 20 and 820 °C inclusive were examined. The rate of heating was 4 °C/min up to approximately 400 °C whereupon it decreased gradually. The required temperature was held for 1 h followed by 24 h of cooling. The concrete strength was 38.6 N/mm² and testing was done after 28 d.

Small 70 mm cubes were tested for the residual concrete strength. The results are shown in Table 3. A point to be noted is that, for room temperature conditions, only one out of the four specimens failed in bond. For the others the steel failed in tension. This means that the bond strength at 20 °C (i.e., the datum or 100% value) can only be tentatively estimated and must be an underestimate. It can be seen that the bond deteriorates with heating and at a greater rate than the concrete compressive strength, and probably even faster than the figures indicate because of the underestimate of the 20 °C bond strength.

(4) Investigations in this field have been carried out in Iran by Ghahramani and Sabzevari [16]. Six inch (153 mm) cubes were used with the steel bar projecting from both ends of the concrete (Fig. 6(d)). Plain and deformed bars were employed along with two rates of heating, i.e., a slow and a fast cycle.

For the slow cycle test the complete specimen was heated over a period of 24 h ensuring that the steel and concrete remained at approximately the same temperature. With the fast cycle, only the two bar ends were heated for 4 h, causing the steel to attain a higher temperature than the concrete. The maximum temperature in both cases was 150 °C with time allowed for cooling to room temperature before loading to failure. The tests were carried out at an age of 28 d and a concrete strength of 21.4 N/mm² was used. Bond strength measurements were taken at a loaded end slip of 0.005 in. (0.127 mm) and also at the complete pull-out state. Similar specimens were made for checking the concrete compressive strength after cooling from 150 °C.

The results are given in Table 4 where,

\[
\text{bond ratio} = \frac{\text{bond strength after cooling}}{\text{compressive strength after cooling/}} \frac{\text{bond strength at 20 °C}}{\text{compressive strength at 20 °C}}
\]

The bond ratio indicates the effect of a change in thermal conditions on bond with concrete of a given strength.

The results indicate that at 150 °C the bond strength is adversely affected to a greater extent than the concrete compressive strength.

The conclusion reached by Ghahramani and Sabzevari was that the bond quality is reduced when subjected to elevated temperature and that this reduction is greater for a fast heating cycle than for a slow one and for deformed as opposed to plain bar.

The bond ratio emphasises which property, bond or compressive strength, is more affected by a change in thermal conditions.

(5) Tests on the bond strength between steel and refractory concrete have been under-
TABLE 3
Variation of concrete compressive and residual concrete–steel bond strength with temperature according to Morley [15]

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>20</th>
<th>300</th>
<th>500</th>
<th>650</th>
<th>800</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum bond strength (percentage of the 20 °C value)</td>
<td>100</td>
<td>69</td>
<td>54</td>
<td>46</td>
<td>34</td>
</tr>
<tr>
<td>Compressive strength of concrete</td>
<td>200</td>
<td>85</td>
<td>96</td>
<td>88</td>
<td>45</td>
</tr>
</tbody>
</table>

TABLE 4
Variation of concrete compressive and residual concrete–steel bond strength with temperature according to Ghahramani and Sabzevari [16]

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Slow cycle</th>
<th>Fast cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive strength</td>
<td>20</td>
<td>150</td>
</tr>
<tr>
<td>Deformed bar</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum bond stress (percentage of the 20 °C value)</td>
<td>100</td>
<td>122</td>
</tr>
<tr>
<td>Bond ratio</td>
<td>100</td>
<td>86</td>
</tr>
<tr>
<td>Plain bar</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum bond stress</td>
<td>100</td>
<td>99.5</td>
</tr>
<tr>
<td>Bond ratio</td>
<td>100</td>
<td>81</td>
</tr>
</tbody>
</table>

taken in Russia by Milovanov and Pryadko [17]. They worked with refractory water glass (sodium silicate) concrete prisms of length 300 mm and cross-section 140 x 140 mm (Fig. 6(e)). Plain and deformed bars of 20 mm dia. were tested over the complete embedment length of 300 mm. Temperatures of 100, 250, 350 and 450 °C were considered. The rate of temperature rise was 35 - 75 °C/h with the specified temperature being held for 2 h. Prior to the start of the heating cycle the specimens were dried at 100 °C for 32 h.

Testing took place at a specimen age of 2 months, and an ambient room temperature concrete strength of 28.6 N/mm², the concrete strength being obtained from 100 mm test cubes after standard curing. On heating to 100 °C the bond strength increased by 36% with heating beyond that having little additional effect. Tests were done in the hot and cooled down states and the results were compared with previous work done on the bond between steel and refractory Portland cement concrete.

The results in the cooled down state, Table 5, are in stark contrast to the other work that has been studied. In this case the bond strength increases with the initial rise in temperature and gradually falls as the temperature continues to increase.

This behaviour is explained in the paper as being the result of the increase in the concrete strength and the shrinkage of the cement paste onto the bar ensuring friction and surface adhesion as heating commences. These are eventually outweighed by the difference in the thermal deformations of the steel and concrete, and so the bond strength is reduced as the temperature continues to increase.

The conclusions drawn from this work included the facts that considerable bond is available for temperatures up to 450 °C, and that the bond of plain and deformed bars to refractory water glass concrete is almost the same in the heated or the cooled down state, the heated state giving the slightly lower values. Also, it was noted that consideration of the bond between steel and concrete is necessary when designing for fire resistant reinforced concrete structures.

(6) By comparison, some other Russian work reported by Bushev et al. [18] indicated that for ordinary reinforced concrete using plain and deformed steel bars the residual bond strength, following heat treatment in the temperature range 20 - 450 °C, did vary. For plain bars the residual bond strength decreased as prior heat treatment temperature level increased. The use of deformed bar, on the other hand, resulted in an increase in resi-
TABLE 5
Variation of residual concrete–steel bond strength with temperature for refractory concrete according to Milovanov and Pryadko [17]

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>20</th>
<th>100</th>
<th>250</th>
<th>350</th>
<th>450</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum bond strength</td>
<td>Water glass concrete</td>
<td>Deformed bar</td>
<td>100</td>
<td>110</td>
<td>106</td>
</tr>
<tr>
<td>(percentage of the 20 °C value)</td>
<td>Plain bar</td>
<td>100</td>
<td>106</td>
<td>102</td>
<td>97</td>
</tr>
<tr>
<td>Refractory Portland Cement concrete</td>
<td>Deformed bar</td>
<td>100</td>
<td>170</td>
<td>160</td>
<td>144</td>
</tr>
<tr>
<td>Plain bar</td>
<td>100</td>
<td>140</td>
<td>116</td>
<td>62</td>
<td>28</td>
</tr>
</tbody>
</table>

TABLE 6
Variation of residual concrete–steel bond strength with temperature reported by Bushev et al. [18]

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>20</th>
<th>100</th>
<th>200</th>
<th>300</th>
<th>450</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum bond strength</td>
<td>Deformed bar</td>
<td>100</td>
<td>108</td>
<td>112</td>
<td>106</td>
</tr>
<tr>
<td>(percentage of the 20 °C value)</td>
<td>Plain bar</td>
<td>100</td>
<td>70</td>
<td>45</td>
<td>23</td>
</tr>
</tbody>
</table>

dual bond strength with temperature for prior heat treatments up to 200 °C and a decrease thereafter for previous thermal exposure in the range 200 - 450 °C. Precise details of the specimen size, concrete, and steel quality, heating cycle, etc. were not given. The results are summarised in Table 6.

(7) Some preliminary investigations by Hertz [19] demonstrate that bond failure of concrete components reinforced with deformed bar and having various geometries can be predicted for temperature levels up to nominally 400 °C, above which specific testing is required.

(b) High temperature bond strength
Preliminary work by Schneider and Diedrichs [20] considered the bond strength at high temperature. Two test methods were used, namely, no applied stress during the heating-up period followed by loading at the temperature under consideration, and constant load applied throughout the complete heating cycle.

Cylindrical specimens of 176 mm dia. (approximately) and length 192 mm with a bond length of 80 mm were used along with 16 mm dia. deformed reinforcing bars (Fig. 6(f)).

For these experiments as well as obtaining the bond stress against temperature relationship, the bond slip versus bond stress curve for different temperatures was established, the stress and slip being measured continuously throughout a test.

(i) For the tests with no stress applied during heating-up, the temperature was held constant at the required level for three hours before commencement of loading. The results, Table 7, show a decrease in bond strength corresponding with an increase in temperature.

TABLE 7
Concrete–steel bond strength at elevated temperatures according to Schneider and Diedrichs [20]

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>20</th>
<th>300</th>
<th>500</th>
<th>680</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum bond strength</td>
<td>100</td>
<td>90</td>
<td>50</td>
<td>28</td>
</tr>
</tbody>
</table>

In the report these values were compared with concrete cylinder compression strength results which indicated that the two are related to temperature in almost the same way.

(ii) Where constant loading was applied throughout the heating cycle, the greater the loading the lower the temperature reached when bond failure took place, e.g., at 60% of the maximum load obtained at 20 °C, bond failure occurred at approximately 125 °C whereas for 20% of this datum load failure occurred at 560 °C. It should be noted that
the maximum load that bond could withstand at 20 °C was far in excess of the code requirements of CP110 Table 21 [21], 3.4 N/mm² ultimate local bond stress for grade 40 concrete or better. For such a bond strength these test results indicate that a temperature ~600 °C could be sustained prior to bond failure. The test results are very encouraging from a design point of view but require the support of more experimental evidence.

GENERAL COMMENTS ON BOND WORK AT ELEVATED TEMPERATURES

(a) Rate of temperature rise: One of the main differences in the various experiments is the rate of temperature rise. Part of the reason for this is that there are two ways of approaching this point of experimental procedure.

(i) A fast rate of temperature rise can be adopted, e.g., the standard test fire [22]. This can be used to produce conditions similar to those found in practice. If the standard test fire is used then comparisons with other fire tests are possible, and results can be related to the length of fire resistance. Reichel [14] used this approach.

(ii) Alternatively a much slower rate of temperature rise can be adopted as used by Harada et al. [13]. This way a gradual temperature rise over the whole specimen is achieved preventing the occurrence of large temperature gradients throughout the section. This eliminates the mechanical stresses caused by differences in thermal expansion between the outside and the centre of the concrete. Thus the results obtained show more clearly the effect on bond of the changes caused by temperature alone.

(b) Compressive strength of concrete: Figure 7 shows that the results of the different experiments were widely divergent. Data from Harada et al. [13] follow roughly the basic recognized curve [4, 5], cf. Fig. 1. Both Ghabra- mani/Sabzevari [16] and Milovanov/Pryadko [17] demonstrated significant increases in strength at 100 - 150 °C, while Morley [15] showed that the strength at 500 °C was almost the same as that at ambient conditions. Varying values can to some extent be explained by the use of different types of cement and aggregate, e.g., Milovanov and Pryadko [17] used refractory aggregate and water glass cement.

A further difference could arise from the differing size and shape of the specimen, e.g., both cubes and cylinders were used.

(c) Experimental approach: It is interesting to note the various experimental procedures. They were all along the lines of the pull-out test, but there appeared to be the same progression as found in tests at ambient temperature [7 - 12]. The specimen types cover a range from those with a long embedment length [13 - 17] with slip measured at the ends only, to those with a small bond length for which end slip is an indication of the breakdown in local bond [20]. Beam tests for bond at elevated temperatures do not appear to have been undertaken, probably because of a desire to keep tests simple, e.g., beam-test specimens would be more complex and involve the expense of using electric resistance strain gauges at elevated temperatures.

(d) Bond strength: Figure 8 shows the residual bond strength results mentioned above. Although they have all been plotted, a direct comparison is not possible. This is due to the differing experimental procedures employed. In order to compare the results, the same conditions would need to prevail for all the tests, whereas, in reality, the differences are many — concrete strength; type and size of bar; age of specimen at test; size and shape of specimen; rate of heating. It is this that accounts for the wide scatter in the various curves, and even the contradictory nature of some of the
results, e.g., Reichel [14] shows the plain bar giving a greater reduction in bond strength compared with the deformed bar whereas with Ghahramani and Sabzevari [16] the situation is reversed.

However, the one point that does become evident from a study of this work is that the reduction in bond due to temperature increase could be a critical factor when assessing the strength of heat-treated, reinforced concrete members.

A summary of the above results shows that Harada et al. [13], Morley [15], and Ghahramani and Sabzevari [16] found the bond quality deteriorating more rapidly than the concrete compressive strength, whilst Reichel [14] concluded that the residual bond strength reduces at a greater rate than the residual steel strength. On the other hand, Milovanov and Pryadko [17] state that for refractory concrete there is considerable bond strength up to 450 °C although the need to consider the effects on bond when dealing with elevated temperatures is emphasised.

Consequently, a number of tests have been carried out which give an indication of the effect of temperature on the bond strength, but comparisons are difficult. They also give information on only a limited number of parameters. For instance, Reichel [14] considered two types of bar and concrete strength, and Ghahramani and Sabzevari [16] looked at two different heating cycles. However, variables affecting the bond are numerous. On top of this the number of results obtained in the above tests are too small to enable an accurate picture of a bond strength–temperature relationship to be obtained. Owing to the variable nature of concrete, which can lead to a wide scatter of results even under similar test conditions, a substantial amount of data are required to be able to produce a reliable bond–temperature relationship. All this adds up to the fact that the work mentioned above is only the beginning of research into a subject about which little is known.

**RELEVANCE TO STRUCTURAL PERFORMANCE**

In reinforced concrete members, adequate bond between steel and concrete is essential for ensuring a satisfactory structural performance. The bond strength, when using plain, hot-rolled steel bars, depends very much on adhesion and frictional resistance. However, with the adoption of deformed steel bars in reinforced concrete construction the factors, on which primary reliance for bonding is placed, become that of bearing between the concrete and the ribs, and the strength of the concrete between the ribs. The mechanism of bond failure in beams and simple pull-out specimens under these latter circumstances is characterised by longitudinal splitting of the surface nearest the reinforcing bar prior to the occurrence of free end slip [23]. This type of failure is much more sudden than with smooth bars and can occur with little warning.

It follows that the effects of temperature on the bond strength is an important factor when assessing (a) the fire resistance or (b) the residual strength of a member.

(a) At the moment the code (CP110: Part 1: 1972) [21] gives a guide to fire resistance in the form of minimum values of the width or depth of the member and the cover to the reinforcing steel. It is an arbitrary method based on fire test results and does not take into consideration the individual circumstances involved. However, in many cases a satisfactory solution is obtained [1].

A different approach to the problem is to design the required fire resistance into the member by analytical means based on known heat transfer and material property behaviour. This is termed ‘rational design’. Briefly stated
it involves obtaining the temperature distribution throughout the member section, then, knowing the effect of temperature on the material properties, estimating the structural strength for the given conditions. The required fire resistance can be obtained by varying the loading, amount of steel reinforcement, concrete cover to reinforcement, and type of concrete used [24, 25]. The type of member is also of importance—continuous beams and slabs giving better fire resistance than simply supported ones under the same conditions, due to their ability to redistribute bending moments. The restraint of thermal expansion can be crucial, for if the surrounding part of a structure is able to withstand the additional thrust caused by the expansion of a component, an external prestressing effect is set up.

In order to be able to design for fire resistance in this way it is essential to have an understanding of the properties of reinforced concrete at elevated temperatures.

(b) When confronted with a fire-damaged building an assessment has to be made of each individual member to decide whether it is suitable for reinstatement or not. The aim when reinstating a member is to restore it to a condition in which it will be able to carry the applied loading. In order to achieve this it is sometimes necessary to provide structural as opposed to cosmetic repair. The basic procedure is [4],

(i) assess the strength of the member remaining after the fire,
(ii) compare the remaining strength with the actual strength required,
(iii) if the remaining strength is less than that needed, add reinforcement and concrete capable of sustaining the difference; if it is greater, then a cosmetic repair only is required.

When assessing the residual strength of a member, once again knowledge of the material properties under such conditions is necessary.

From (a) and (b) above it is clear that the material properties under the respective conditions must be known, i.e., for fire resistance design it is the properties at elevated temperature, whereas for reinstatement it is the residual characteristics that are needed. In both cases the behaviour of concrete compressive and steel tensile strengths have been investigated and with this information good estimates of the behaviour can be made for the two conditions. However, the bond strength is also of importance when considering the structural performance of reinforced concrete. Therefore, to gain a more precise insight into high temperature and residual behaviour of reinforced concrete members, it is necessary to examine the bond quality under these conditions as well.

The tests to determine bond strength both at elevated temperatures and in the residual ambient condition have been of a static nature. No attempt has been made to study the effects on bond strength of fluctuations in load under these conditions, e.g., effects of variations in superimposed loading. It is quite possible that some degradation of bond strength, both residual and at elevated temperature, could occur under variable repeated loading conditions even of a quasi-static nature. The needs of design codes are for information from which reliable long term bond strength values could be specified for the residual, post-fire situation of a structure as well as conservative values for use in fire resistant design. The results obtained so far are insufficient in number and even when suitably augmented they will require factoring down for design purposes.

The simple pull-out type of test provides a basis for the understanding of tests in the flexural situation although it is from the latter that information on the bond strength of bar groups in bending elements subjected to fire will be obtained.

FUTURE WORK

In an effort to improve the state of knowledge on this topic the authors are preparing to investigate more fully the characteristics of the residual bond quality in order to establish its significance when assessing the structural strength of fire damaged reinforced concrete and designing for its reinstatement.

The type of specimen to be used is shown in Fig. 9. It is along the lines of Rehm [11], which should yield a direct bond stress-slip result.

Four specimens will be tested together in a small electric furnace, the load applied and the slip between the steel and concrete being continuously measured by load cells and dis-
placement transducers, respectively. The test apparatus is shown in Fig. 10.

The slip is measured at the end of the specimens and so a correction has to be made for the difference in thermal expansion between the length of unbonded steel and concrete.

With this equipment it is planned firstly to do a series of residual bond tests under the no stress condition during the heating cycle. By monitoring movements throughout the heating cycle when no load is applied, the information needed to make the correction for the difference in thermal expansion between the length of unbonded steel and concrete will be obtained. In an attempt to simulate actual conditions more realistically tests will be carried out in which a constant load is applied throughout the heating cycle, until cooling to room temperature has taken place. Then the load will be increased to failure.

These results will be compared with the initial test series and also with a series in which the load is applied after the specimen has reached the specified elevated temperature, which will be held constant until failure occurs. A similar series of tests to the latter will be performed with a working load applied throughout the heating cycle.

Various parameters will be considered, e.g., aggregate size and type, bar size and type, cement type, cover, aggregate/cement ratio, water/cement ratio, moisture content at time of test, direction of casting, temperature, heating and cooling rates, etc. Also the effects of working stress level and repeated loading on bond strength will be studied.

A series of tests giving the concrete compressive strength will be carried out enabling a comparison to be made with bond strength under the same conditions.

Further useful work is being contemplated along the lines of testing small flexural specimens. Such information could be valuable in checking the accuracy of the general results obtained from the pull-out tests for the specific case of a beam under a given set of conditions. The furnace described for the bond pull-out tests could be modified for this purpose.

Another avenue of investigation is the development of a non-destructive method of assessing the residual bond strength after a fire, e.g., by use of ultrasonics or acoustic emission. By examining the crack pattern at the concrete/steel interface prior to heat treat-
ment of the bond pull-out specimens, and after heat treatment but before determination of residual bond strength, it could be possible to establish a correlation between crack pattern deterioration and residual bond strength. Such a correlation would be most useful when examining actual fire damaged structures by the same non-destructive technique.

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APPENDIX 2

Normal Pressure Under Ribs
Expressions for evaluating the normal pressure on the concrete under the ribs of a deformed bar have been deduced by Rehm (33). These expressions have been adapted below to estimate the prevailing order of magnitude of the pressure on the concrete under the ribs at the critical value in this work.
CALCULATION FOR 20°C

Referring to Fig. 40

nominal stress in the concrete under ribs \( \sigma_{cn} \) = \( \frac{\text{force from ribs}}{\text{rib area projected on a normal plane}} \)

where force from ribs = total force - force due to applied sliding resistance of bar surface

\[ P = \text{total force applied} \]
\[ l_v = \text{bond length} \]
\[ a = \text{rib height} \]
\[ n = \text{no. of ribs} \]
\[ \alpha = \text{helix angle} \]
\[ u = \text{mean bar perimeter} \]

\[ \therefore \sigma_{cn} = \frac{P - \sigma_{bp} \cdot u \cdot l_v}{n \cdot a \cdot l_v \cdot \cot \alpha} \]

(a) Force from ribs

(i) Total force applied

\[ P = \sigma_b \cdot u \cdot l_v = 5.39 \times \pi \times 15.93 \times 32 \]
\[ = 8635.3 \text{ N} \]

where \( \sigma_b \) = bond stress of deformed bar at critical value for 20°C (cf. Fig. 38).

(ii) Force due to sliding resistance of bar surface

\[ \sigma_{bp} \cdot u \cdot l_v = 3.28 \times \pi \times 15.93 \times 32 \]
\[ = 5254.9 \text{ N} \]

where \( \sigma_{bp} \) = bond stress (sliding resistance) at critical value slip of deformed bar for 20°C. Taken from plain bar results in Fig. 67 and adjusted
in accordance with the assumption set down in appendix 3. This adjustment is needed as the $\sigma_b$ values are for a steady state bond stress of $3.70 \text{ N/mm}^2$ whereas the $\sigma_{bp}$ values are for a steady state bond stress of $2.45 \text{ N/mm}^2$.

\[ \text{Force from ribs} = P - \sigma_{bp} \cdot u \cdot l_v \]
\[ = 8635.3 - 5254.9 \]
\[ = 3380.4 \text{ N} \]

(b) **Ribs area projected on a normal plane**

(i) **Transverse ribs**

\[ \text{area} = n \cdot a_1 \cdot l_v \cdot \cot \alpha \]
\[ = 4 \times 0.38 \times 32 \times \cot 41.2 \]
\[ = 55.4 \text{ mm}^2 \]

(ii) **Helical ribs**

\[ \text{area} = n \cdot a_1 \cdot l_v \cdot \cot \alpha \]
\[ = 2 \times 0.975 \times 32 \times \cot 71.3 \]
\[ = 21.2 \text{ mm}^2 \]

**Total rib area** = $55.4 + 21.2 = 76.6 \text{ mm}^2$

\[ \therefore \sigma_{cn} = \frac{3380.4}{76.6} = 44.13 \text{ N/mm}^2 \]

44.13 N/mm$^2$ compares favourably with the cube strength of 35 N/mm$^2$ indicating that the critical point coincided with the start of the compressive failure of the concrete under the ribs. The slightly higher strength obtained is consistent with the smaller area and greater restraint experienced beneath the ribs as compared with the cube strengths. Using the same procedure the $\sigma_{cn}$ values for the elevated temperatures were computed.
APPENDIX 3

The Assumption Used To Obtain The Plot Of $\sigma_{\text{on}}$ - Temperature For Different Test Conditions
Aim

To plot the $\sigma_{cn}$-temperature curve for test conditions for which the deformed bar values are known but the plain bar resistance to sliding is not known.

Assumption

At the critical value of slip the variation in the bond stress for plain bars with test condition is directly, proportional to the variation of bond stress for deformed bars under the same test conditions. The argument justifying this assumption is set out in steps below.

1. $\sigma_{cn}$ depends on the difference between the bond stress for the deformed bar at the critical slip value and the resistance of the smooth portion of the bar at the same critical slip value.

2. The resistance of the smooth portion of the bar is estimated from the plain bar results.

3. The bond stress-temperature curves for both deformed and plain bars follow the pattern of the concrete compressive strength curve (cf. Fig. 29).

4. Therefore the bond stress is directly related to the concrete compressive strength.

5. The experimental investigation shows that a change in the test conditions causes a slight variation in the bond stress-temperature curves to take place for the deformed bars (cf. Figs. 53-57).

6. It is known from other work that changes in test conditions cause a variation in the concrete compressive strength-temperature curve to take place.
(7). It is assumed from (3-6) that the change in the concrete strength under the different test conditions is directly related to the change in bond stress that takes place under those conditions.

(8). From this it follows that the change in the bond performance between the deformed and plain bars will be related to each other via the concrete strength.

(9). Hence assume the change in the plain bar bond stress for different test conditions to be directly proportional to the change in the deformed bar bond stress for those same conditions.

(10). Datum values that were used were the known deformed and plain bar values under the test condition of 2.45 N/mm² steady state bond stress and residual loading to failure.
APPENDIX 4

$\sigma_{cn}$ Values For 600 And 750°C
The same procedure as in appendix 2 above is used. However from the plain bar results (Figs. 46 and 68) it can be seen that when under a steady state stress during heating the bond failed before temperatures of 600 and 750°C were reached. Hence in this case the sliding resistance for the deformed bar at these temperatures is assumed to be zero.

Thus the force from the ribs in the presence of a steady state bond stress of 3.70 N/mm² and a sliding resistance of zero for 600 and 750°C is \( \sigma_{cn} \).

\[
\therefore \quad P = 3.70 \times \pi \times 15.93 \times 32 = 5927.8 \text{ N}
\]

\[
\therefore \quad \sigma_{cn} = \frac{\text{force from ribs}}{\text{projected rib area}} = \frac{5927.8}{76.6} = 77.39 \text{ N/mm}^2
\]

This \( \sigma_{cn} \) value is clearly too high to be consistent with the \( \sigma_{cn} \) values at 20-450°C. The reason is that the decrease in sliding resistance caused the critical slip value to be exceeded during the heating up period. Consequently \( \sigma_{cn} \) does not exist for temperatures of 600°C and above during loading to failure under these test conditions.
APPENDIX 5

Shear Stresses In Tooth Between Ribs
Expressions put forward by Rehm (33) for calculating the shear stresses have been modified for use in this work.

Referring to Fig. 40,

\[ \sigma_{cs} = \frac{P - \sigma_{bp} A_{nr}}{A_r} \]

where:
- \( P \) = total applied force
- \( A_{nr} \) = area not resisting shear
- \( \sigma_{bp} \) = bond stress due to sliding resistance of smooth part of bar
- \( A_r \) = area resisting shear

\( A_{nr} \)

(i) Transverse rib surface area

\[ l_x = \frac{y}{\sin \alpha} \]

where:
- \( l_x \) = the length of the respective ribs over the bond length of 32 mm.
- \( y \) = allowance for where the transverse ribs fade out near the helical ribs.
- \( \alpha \) = helix angle

\[ l_x = \frac{32 - 10.5}{\sin 41.2} = 32.64 \text{mm} \]

(ii) Helical rib surface area

\[ l_x = \frac{y}{\sin \alpha} = \frac{32}{\sin 71.3} = 33.78 \text{mm} \]
total surface area of ribs = \( \Sigma (l \times \text{width} \times \text{no. of ribs}) \)

= \((32.64 \times 2 \times 4) + (33.78 \times 1.5 \times 2) \)

= 362.46 mm\(^2\)

(b) **Area resisting shear** \((A_r)\)

\[ A_r = \text{total surface area} - A_{nr} \]

= \( \pi d l \times 362.46 \)

= 1246.7 mm\(^2\)

(c) **Shear force**

\[ \text{shear force} = P - \sigma_{\text{cr}} \cdot A_{nr} \]

= \( (\sigma_{\text{cr}} \cdot u.l.) - \sigma_{\text{cr}} \cdot A_{nr} \)

= \( (\sigma_{\text{cr}} \cdot 1609.1) - (\sigma_{\text{cr}} \cdot 362.46) \)

\( \sigma_{\text{cr}} \) and \( \sigma_{\text{cr}} \) are obtained from the bond stress-slip curves at a slip value consistent with the end of fracture surface development.

From Fig. 90 \( \sigma_{\text{cr}} = 7.70 \text{ N/mm}\(^2\) at a slip of 0.155 mm

From Fig. 67 at a slip of 0.155 mm \( \sigma_{\text{cr}} = 4.2 \text{ N/mm}\(^2\)  

Hence, shear force = \( (7.70 \times 1609.1) - (4.2 \times 362.46) \)

= 10867.7 N

\[ \sigma_{\text{cs}} = \frac{P - \sigma_{\text{cr}} \cdot A_{nr}}{A_r} \]

= \( \frac{10867.7}{1246.7} \)

= 8.72 N/mm\(^2\)

Now cube strength at 300\(^\circ\)C = 32.55 N/mm\(^2\)

\[ \frac{\sigma_{\text{cs}}}{\sigma_{\text{cu}}} = \frac{8.72}{32.55} = 0.268 \]

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The other points plotted on Fig. 42 are obtained using the same procedure as above.

Note: in the above calculation no adjustment was made to the plain bar values to allow for the differences in steady-state stress of the two bar types. This is because in this calculation the effect it has is negligible.

E.g., at 300°C with 3.70 N/mm² steady-state bond stress and 0.155mm slip, from Fig. 90 \( \sigma_b^- = 7.70 \text{ N/mm}^2 \).

At 300°C with 2.45 N/mm² steady state bond stress and 0.155mm slip, from Fig. 64 \( \sigma_b^- = 8.70 \text{ N/mm}^2 \).

At 300°C with 2.45 N/mm² steady state bond stress and 0.155mm slip, from Fig. 67 \( \sigma_{bp}^- = 4.2 \text{ N/mm}^2 \).

Adjusted for a steady state bond stress of 3.70 N/mm²

\[
\sigma_{bp}^- = 4.2 \times \frac{7.70}{8.70} = 3.72 \text{ N/mm}^2
\]

\[\sigma_{cs}^- = \frac{11041.9}{1246.7} = 8.86 \text{ N/mm}^2\]

\[\frac{\sigma_{cs}^-}{\sigma_{en}^-} = \frac{8.86}{32.55} = 0.272\]
APPENDIX 6

The Behaviour Of Reinforced Concrete
At Elevated Temperatures With
Particular Reference To Bond Strength

by

R. Royles, P.D. Morley and M.R. Khan

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THE BEHAVIOUR OF REINFORCED CONCRETE
AT ELEVATED TEMPERATURES WITH
PARTICULAR REFERENCE TO BOND STRENGTH

R. Royles, P.D. Morley, and M.R. Khan
University of Edinburgh
Department of Civil Engineering and Building Science

ABSTRACT

The influence of elevated temperature on the residual bond between concrete and steel was examined over a range of temperatures from 20 to 800°C. Simple circular cylindrical pull-out specimens were employed with a relatively short bond length. The same type of concrete mix was used throughout and the steel reinforcement was of deformed (i.e. ribbed) bar.

Bond stress–slip relations were examined in detail together with associated acoustic emission data obtained from the interface. Two different bar diameters were utilised, 8 and 16mm, and some indication of the affect of bar size on bond performance was gained.

Stress components in the region of the interface were deduced from the test results and were considered with regard to a mechanism of bond breakdown.

INTRODUCTION

The satisfactory response of reinforced concrete structures to loading depends to a large extent on the strength properties and durability of the individual materials and on the existence of adequate bond between the concrete and steel.

One of the major dangers confronting buildings is the threat of fire and they should be provided with sufficient structural fire resistance to give occupants time to escape before strength and or stability failure ensues.

In many instances reinforced concrete structures remain standing after a fire and some assessment has to be made of its capacity for re-use.

Aspects of design both for fire resistance and post-fire reinstatement in reinforced concrete structures dictate a requirement for knowledge of the quality of the bond between the two constituent materials under conditions of loading both at elevated temperature and in the cooled down state.

A lot of information exists regarding the characteristics of concrete and steel under these conditions but corresponding data on the quality of the bond between the two materials is somewhat sparse.

Recently a review of work in this area showed that it was difficult to correlate the various results because of the differences in test conditions, test procedures, and specimen form. An investigation by Hertz of anchorage failure in post-heat treated reinforced concrete led to a proposed standard method for predicting this failure. The proposal was based on an experimental approach to the determination of bond strength using circular cylindrical pull-out specimens with the pressure applied to the concrete through a 90° conical surface. This was an attempt to transfer compressive stresses to the interface through planes at 45° to the bar axis as might occur in flexural components. However, friction at the conical bearing surface would modify the desired stress pattern.
As part of a programme to provide further fundamental information concerning the influence of temperature on the performance of reinforced concrete structures during and after a fire, tests are reported here on the residual bond behaviour after heat treatment in the range 20 - 800°C. These tests included an examination of the release of elastic strain energy at the interface in the form of acoustic emission (AE) during bond breakdown. Deductions are made from the results giving some insight into the mechanism of bond failure.

**NOTATION**

- **AER**: Acoustic emission ratio—emission as a proportion of the total accumulated emission counts at maximum bond stress.
- **BSR**: Maximum residual bond stress as a proportion of the bond strength of an 8mm dia. bar at ambient temperature.
- **CSR**: Concrete compressive strength as a proportion of the cube crushing strength at ambient temperature.
- **T**: Temperature (°C)
- **Tf**: Furnace temperature (°C)
- **Tm**: Concrete/steel interface temperature (°C)
- **Tmax**: Maximum interface temperature (°C)
- **σb**: Bond stress, deformed bar (N/mm²)
- **σbp**: Bond stress, plain bar (N/mm²)
- **σc**: Compressive stress on concrete beneath a rib and normal to cross section of a bar (N/mm²)
- **σcu**: Concrete compressive cube strength (N/mm²)

**SPECIMEN DESIGN**

A circular cylindrical form of specimen was chosen having a height of 300mm and a diameter appropriate to the symmetrical concrete cover required over a central axial bond length of 32mm, approximately. The reinforcement was a metre length of 8 or 16mm dia. Tor Bar to BS 4461:1978 and classified as type 2 deformed bar. The concrete cover employed was 55mm, nominal, and the bar was flush with the concrete face at one end and protruding from the other.

A 19mm natural aggregate (shrinkage value = 0.07%) was used in the concrete with the coarse and fine parts conforming with the sieve analysis limits of BS 882 : Part 3 : 1973. The fines were consistent with a zone 3 sand. The cement was ordinary Portland and the mix a nominal 1:2:4 with a water:cement ratio of 0.6.

The direction of casting was parallel to the axis of the reinforcement with the protruding bar downwards, and curing took place over 3 months in air at a relative humidity of 60%.

The specimen shape provided a symmetrical radial thermal path and a bond length between two and four bar diameters in an endeavour to simulate local bond stress conditions without being too small in relation to the aggregate size.

**EXPERIMENTAL ARRANGEMENT AND TEST PROCEDURE**

The specimens were mounted vertically in batches of four within a purpose designed electric furnace of 1000 x 250 x 300mm internal dimensions. Each reinforcing bar extended through the base of the furnace to a loading arrangement incorporating a 50 kN load cell. Loading was by a manually operated hydraulic
system, which could apply load simultaneously to all four specimens or individually, as required. The relative slip was monitored by a displacement transducer (LVDT type, ± 2.5 mm range with a linearity of 0.3% of f.s.d.) mounted above a specimen as shown in Fig. 1. A general impression of the test arrangement is given in Fig. 2 showing the instrumentation in the foreground and the furnace in the background with the displacement transducers on top.

In order to avoid steep thermal gradients and to keep the heating cycle within a working day a slow heating rate of 2°C/min was employed; both furnace and interface temperatures were recorded continuously from chromel-alumel thermocouples. All specimens were saturated for one hour at their respective maximum temperatures before cooling slowly to ambient with the furnace switched-off and the lid removed.

Thereafter the specimens were pulled to failure with continuous recording of load and slip. A uniform loading rate was maintained equivalent to a bond stress increase of approximately 3 N/mm² per minute.

During the bond-slip studies the specimens were heated and cooled with zero bond stress applied. However for the examination of AE from the interface, in the case of the specimens with 16mm dia bars a steady-state bond stress was applied. This was equivalent to the ultimate local bond stress consistent with the quality of concrete at ambient temperature, i.e. grade 35 - established at 28 days from cubes and cylinders cast in the same way as the bond specimens.

The AE was monitored by a piezo-electric accelerometer inserted between the displacement transducer and a ceramic wave guide bonded on to the upper end of the reinforcing bar. The wave guide introduced almost negligible attenuation of the signals which were amplified and recorded as accumulating events on a chart recorder via a threshold event counter; the overall gain on the original signals being 72dB. The system was calibrated before and after each test using a standard Nielsen source.

BOND-SLIP RELATIONS

The development of slip with increasing local bond stress in the residual heat treated condition is shown in Figs 3 and 4 for the 8 and 16mm dia bars respectively.

The interface temperature did not saturate at the same level as the furnace or surface temperature no doubt due to differing conditions of thermal equilibrium prevailing in the two locations. The saturation values for each location are marked against the appropriate curves.

The general shape of the curves indicate a more rapid early increase of bond stress with a discontinuity beyond which stress rises at a more gradual and diminishing rate to a maximum for large amounts of slip (~ 6% of the bond length). Thereafter pull-out failure is approached with the bond stress decreasing at an accelerating rate.

For each bar size the bond strength tended to be weaker than at ambient for temperatures between 20 to 100°C and then some increase occurred between 100 to 300°C and even as high as 375°C for the 8mm dia.bar. Subsequently bond strength diminished with temperature. This is demonstrated clearly in Fig 5 where the maximum residual bond strength is plotted against interface temperature.

It is interesting to note that the variation in maximum residual bond strength with temperature, Fig 5, follows a very similar pattern to that of the crushing strength, shown in Fig 6. In the main similar patterns to that of Fig 5 pertain at different proportions (< 100%) of the maximum residual bond strength.
Comparing the performance of 8 and 16 mm dia bars it is evident from Figs 3 and 4 and particularly Fig 5 that the smaller bar had the better bond qualities over the whole range from 20 - 800°C.

In some work at elevated temperature Diederichs and Schneider 7 found no significant influence of bar size on bond strength in relation to plain bars but no comparative work on deformed bars was reported. Under ambient temperature conditions Watstein and Bresler 8 felt that bond strength would vary inversely with bar size, a view which supported the earlier findings of Rehm 9.

The apparent inferior bond strength of the larger bar might be attributable in part to the greater tendency for concrete splitting to occur the larger the bar size. Some further appreciation of this point might be gained from a consideration of the failure mechanism.

MECHANISM OF BOND BREAKDOWN

As mentioned above the bond-slip curves, Figs 3 and 4, display two stages of slip. In the first, slip is small up to a discontinuity followed by the second or large slip process.

The geometrical features of a deformed bar are illustrated in Fig 7 and the stress components in the vicinity of the concrete/steel interface are shown in Fig 8a.

An explanation of the apparent two stage slip process could be that initially no local crushing of the concrete under a rib has occurred. Once it has taken place large slip would ensue as fracture surfaces developed beneath the ribs, see Fig 8b. Further increase in bar tension would result in the bond stress rising and wedges of concrete attached to the bar being pulled against adjacent sound material setting up splitting forces. Therefore an accelerating rate of slip could be expected.

In order to try and validate this proposed mechanism of bond failure the normal stress, \( \sigma_{cn} \), in the concrete beneath a rib could be computed in a manner suggested by Rehm 9 from,

\[
\sigma_{cn} = \frac{\text{force from ribs}}{\text{Helical rib surface projected on a normal plane}}
\]

where,

- \( \text{force from ribs} = (\text{total applied force})-(\text{force due to sliding resistance of bar surface}) \)

The sliding resistance could be determined using the bond stress, \( \sigma_{bp} \), on a plain bar surface.

The development of \( \sigma_{bp} \) with slip following heating at various temperatures was found for 16 mm dia plain bar separately 10.

Considering conditions at the discontinuities in the bond-slip curves, Figs. 3 and 4, the corresponding value of \( \sigma_{bp} \) was found for the same slip and hence \( \sigma_{cn} \) was calculated. Due allowance was made for the transverse ribs as well as the longitudinal helical ones. The variation of \( \sigma_{cn} \) with temperature is shown in Fig 6 together with the associated variation of concrete crushing strength, \( \sigma_{cu} \). The pattern of variation of \( \sigma_{cn} \) and \( \sigma_{cu} \) with temperature was very similar. The comparison suggests that at the discontinuity in a bond-slip curve a compressive stress could exist under the ribs greater than the corresponding cube strength. This might be expected since under a rib the material is much more confined and the stress more localised than in a cube test. Under such a high local compressive stress crushing could occur producing mortar particles that would be compressed into adjacent pores giving rise to stress con-
centrations and inducing further failure of the mortar. Principal stresses in
the material between projecting ribs would build up in the remaining sound con-
crete until more cracking/splitting occurred and finally the bond would be de-
stroyed.

ACOUSTIC EMISSION FROM THE BOND INTERFACE

The A.E from the concrete/steel interface is presented in Figs 9 and 10 both in relation to slip and bond stress.

The emission was caused by breakdown of adhesion and cracking along the interface and associated local crushing under the ribs. No other degradation of the concrete was occurring during these tests and the tensile stress in the steel was well below yield.

As can be seen the emission continued throughout a test growing with slip and eventually tending to infinity as slip became very large and extraction was approached. Therefore it is not surprising that the $\sigma_b$ - AE relation is of a similar form to that between bond stress and slip. The amount of information available in the form of counts of significant events i.e. those producing a signal above the 1 volt level at the counter, was large irrespective of whether or not a steady state bond stress was applied. This would imply that critical slip might be identified very easily using AE since $48$dB of unused gain was available within the system for use in the small slip range. A stepping down of the gain could be achieved without difficulty or impairment of accuracy when dealing with larger slip.

The shape of the AE-slip relations were similar for each bar size and typical of the ones tested but were not directly comparable because of the presence of a steady state stress during the heating and cooling cycle in the case of the 16mm dia. bar specimen which was pulled to failure from that stress level.

DISCUSSION AND CONCLUDING REMARKS

The tests have indicated a variation of residual bond strength with temper-
ature similar in appearance to that of plain concrete. The performance of the smaller bar was superior to that of the larger.

There would appear to be a critical point in the slipping process associ-
ated with the onset of local crushing of the concrete beneath the ribs of deformed bar. This point might be identified more readily using AE which would suggest that AE could be a useful parameter in studying the integrity of reinforced concrete components and structures. Such critical points do not appear to have been detected in the study of bond performance with deformed bar at elevated temperature.

Further work is in hand to extend the investigations to bond performance at high temperature, including the influence of concrete cover as well as factors such as bar type and size, aggregate type, water-cement ratio, cement type etc on bond strength. The role of AE monitoring is being extended to the study of components in flexure and shear with the aim, ultimately, of devising a routine for assessing the integrity of fire damaged r.c. structures.

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REFERENCES


Fig. 1  A section of the furnace

Fig. 2  Furnace and associated instrumentation
Fig. 3  Residual bond stress v slip for various temperatures in °C (8mm dia. bar)

Fig. 4  Residual bond stress v slip for various temperatures in °C (16mm dia. bar)
Fig. 5 Maximum residual bond stress v interface temperature (8 and 16mm dia. bar)

Fig. 6 Variation of concrete compressive strength characteristics with temperature (16mm dia. bar)
Fig. 7  Geometry of reinforcement

Fig. 8  Stress components and fracture surfaces in the region of concrete/steel interface.
Fig. 9. Development of AE with residual bond stress and slip for 8mm dia. bar.

$T_i = 250^\circ C; \sigma^t_b = 0$ during heating cycle

At $\sigma^t_b_{\text{max}}$ number of AE counts = 78000
Fig. 10  Development of AE with residual bond stress and slip for 16mm dia. bar.

\( T_{i, \text{max}} = 250^\circ C; \sigma_{b, \text{max}} = 3.70 \text{N/mm}^2 \) during heating cycle

At \( \sigma_{b, \text{max}} \) number of AE counts = 42350
APPENDIX 7

Computer Program
C PROGRAM - ANCHORAGE, DATUM

C ARITHMETIC FUNCTION STATEMENTS

FB(A3, A2, A1, A0, DB) = (A3*DB**3) + (A2*DB**2) + (A1*DB) + A0
FC(A3, A2, A1, A0, DC) = (A3/4.0)*DC**4 + (A2/3.0)*DC**3 +
   (A1/2.0)*DC**2 + A0*DC
FF(FC, A) = 1.0/SORT(FC+A)

C READ IN DATA

WRITE(6,5)
   5 FORMAT(19H A3 A2 A1 A0, 4F12.4)
READ(5,10)A3, A2, A1, A0
10 FORMAT(4F12.4)
WRITE(6,15)A3, A2, A1, A0
15 FORMAT(1X,4F12.4)

WRITE(6,20)
   20 FORMAT(19H DO DCRIT GA, 3F12.4)
READ(5,25)DO, DCRIT, GA
25 FORMAT(3F12.4)
WRITE(6,30)DO, DCRIT, GA
30 FORMAT(1X,3F12.4)

WRITE(6,35)
   35 FORMAT(21H DTRAN GB C D3, 4F12.4)
READ(5,40)DTRAN, GB, C, D3
40 FORMAT(4F12.4)
WRITE(6,45)DTRAN, GB, C, D3
45 FORMAT(1X,4F12.4)

C READ IN CONSTANTS

E=200000.0
DIA=0.0

C CALC CONSTANTS

RKA=SORT((4.0*GA)/(E*DIA))
RKB=SORT((4.0*GB)/(E*DIA))
RK1=SORT((4.0/(E*DIA))]

C PRINT INITIAL SLIP VALUE

11 WRITE(6,12)DO
12 FORMAT(23H VALUE FOR INITIAL SLIP, 17X, F12.4)

C INITIAL SO VALUE

SO=1.0

C TO OMIT STAGE I

IF(DO.GE.DCRIT)GO TO 24

C FOR CORRECT SO VALUE

SO=DO*RKA*RKA*E

C PRINT STRESS VALUE

16 WRITE(6,17)SO
17 FORMAT(17H VALUE FOR STRESS, 23X, F12.4)

C PROGRAM - CRACK

297
C CALC DA SA BA XA FOR STAGE 1
CALL LINE0(D0,SO,E,RKA,DCRIT,GA,SA,BA,DA,XA)

C CALC D1 S1 B1 X1 FOR STAGE 1
CALL LINE1(D0,SO,E,RKA,DCRIT,GA,SA,BA,DA,XA)
D1=DA
X1=XA
B1=BA
S1=SA
WRITE(6,50)D1,X1,S1,B1
50 FORMAT(12H D1 X1 S1 B1,10X,4F12.4)

FOR CASES USING STAGE 1
GO TO 22

FOR CASES OMITTING STAGE 1
24 IF(D0 .GE. DTRAN)GO TO 26
D1=DO
S1=(GB*D1+C)*RK1*RK1*E —— DELETE AND INSERT —— S1=SO
X1=0.0
WRITE(6,29)S1
29 FORMAT(21H VALUE FOR STRESS(S1),19X,F12.4)

C CALC D2 X2 S2 B2 FOR STAGE 2
22 CALL LINE2(D1,S1,E,C,RKB,DTRAN,GB,SB,BB,DB,XB)
D2=DB
X2=X1+XB
S2=SB
B2=BB
WRITE(6,55)D2,X2,S2,B2
55 FORMAT(12H D2 X2 S2 B2,10X,4F12.4)

FOR CASES USING STAGE 2
GO TO 27

FOR CASES OMITTING STAGE 2
26 D2=DO
FB2=FB(A3,A2,A1,A0,D2) —— DELETE AND INSERT —— S2=SO
S2=FB2*RK1*RK1*E
X2=0.0
WRITE(6,36)S2
36 FORMAT(21H VALUE FOR STRESS(S2),19X,F12.4)

C CALC DC XC SC BC FOR STAGE 3
27 FC2=FC(A3,A2,A1,A0,D2)

C CALC CONSTANT A
A=((S2/(SORT(2.0)*RK1*E))992)-(FC2)

C PRINT A
WRITE(6,31)A
31 FORMAT(12H RESULT OF A,3X,F12.5)
C CALC FUNCTIONS FF2 AND FFC
  FF2=FF(FC2,A)
  DC=D2+0.001
  28 FCC=FC(A3,A2,A1,A0,DC)
  FFC=FF(FCC,A)

C CALC INITIAL AREA AND DISTANCE ESTIMATE
  AREA1=(((FF2+FFC)/2.0)*(DC-D2))
  XCI=AREA1/(SORT(2.0)*RK1)

C INCREASE NO. OF SEGMENTS
  N=4
  33 IN=N

C CALC VALUE FOR H
  H=(DC-D2)/FLOAT(IN)

C DO LOOP TO CALC 4*Y VALUES
  FFSEG1=0.0
  DO 32 J=1,N,2
     DSEG1=D2+(FLOAT(J)*H)
     FCSEG1=FC(A3,A2,A1,A0,DSEG1)
     FFSEG1=FFSEG1+(4.0*FF(FCSEG1,A))
  32 CONTINUE

C DO LOOP TO CALC 2*Y VALUES
  FFSEG2=0.0
  NE=N-2
  DO 34 K=2,NE,2
     DSEG2=D2+(FLOAT(K)*H)
     FCSEG2=FC(A3,A2,A1,A0,DSEG2)
     FFSEG2=FFSEG2+(2.0*FF(FCSEG2,A))
  34 CONTINUE

C CALC IMPROVED AREA AND DISTANCE ESTIMATES
  AREA2=(H/3.0)*(FF2+FFSEG1+FFSEG2+FFC)
  XC=AREA2/(SORT(2.0)*RK1)

C CALC ERROR
  ERROR=ABS(XC-XCI)
  ERROR=(ERROR/ABS(XC))*100.0
  N=N+1000
  AREA1=AREA2
  XCI=XC
  IF(ERROR.GT.0.1)GO TO 33

C CALC DISTANCE
  XC=X2+XC

C CALC STEEL STRESS
  SC=SORT(2.0)*RK1*E*SORT(FCC+A)

C CALC BOND STRESS
  FBC=FB(A3,A2,A1,A0,DC)
  BC=FBC

299
C PRINT VALUES
WRITE(6,65)DC,XC,SC,BC
65 FORMAT(12H DC XC SC BC,10X,4F12.4)

C REPEAT PROCESS FOR RANGE OF DC VALUES
DC=DC+0.03
IF(DC.LE.D3)GO TO 28

C LOOP FOR DO VALUES LOOP FOR SO VALUES IF(DO.LT.0.0005)GO TO 70
DELETE
AND INSERT
C LOOP FOR SO VALUES
SO=2.0*SO
IF(SO.LT.300.0)GO TO 11

70 STOP
END

SUBROUTINE LINEO(DO,SO,E,RKA,DCRIT,GA,SA,BA,DA,XA)

X=0.0
5 TERM1=(0.5*(DO+SO/(E*RKA)))*EXP(RKA*X)
TERM2=(0.5*(DO-SO/(E*RKA)))*EXP(-RKA*X)
DX=TERM1+TERM2
IF(DX.GT.DCRIT)GO TO 10
SA=E*RKA*(TERM1-TERM2)
BA=GA*DX
DA=DX
XA=X
WRITE(6,15)DA,XA,SA,BA
15 FORMAT(12H DA XA SA BA,10X,4F12.4)
IF(X.GE.100.0)GO TO 16
IF(X.GE.40.0)GO TO 17
X=X+10.0
GO TO 18
17 X=X+20.0
GO TO 18
16 X=X+50.0
18 GO TO 5
10 RETURN
END
SUBROUTINE LINE1(DO, SO, E, RKA, DCRIT, GA, SA, BA, DA, XA)

X = 0.0
5 TERMI = (DO/2.0 + SO/(2.0*E*RKA))*EXP(RKA*X)
TERM2 = (DO/2.0 - SO/(2.0*E*RKA))*EXP(-RKA*X)
DX = TERMI + TERM2
IF(DX .GT. DCRIT) GO TO 10
SA = E*RKA*(TERMI - TERM2)
BA = GA*DX
DA = DX
XA = X
X = X + 0.01
GO TO 5
10 RETURN
END

SUBROUTINE LINE2(DI, S1, E, C, RKB, DTRAN, GB, SB, BB, DB, XB)

X = 0.0
5 TERMI = (DI/2.0 + C/(2.0*GB) + S1/(2.0*E*RKB))*EXP(RKB*X)
TERM2 = (DI/2.0 + C/(2.0*GB) - S1/(2.0*E*RKB))*EXP(-RKB*X)
DX = TERMI + TERM2 - C/GB
IF(DX .GT. DTRAN) GO TO 10
SB = E*RKB*(TERMI - TERM2)
BB = (GB*DX) + C
DB = DX
XB = X
X = X + 0.01
GO TO 5
10 RETURN
END
APPENDIX 8

Estimation Of Young's Modulus Of Concrete Immediately Beneath Ribs For Stressed And No-Stressed Test Conditions
To calculate the E value for concrete for actual slip and apparent slip (i.e., no-stress and stressed conditions respectively).

At Ambient Temperature

In CP110(23) the value of E for concrete is given as \(5.5\sqrt{\sigma_{cu}}/\gamma_m\) kN/mm\(^2\). If the partial safety factor is taken as unity,

\[ E_c = 5.5\sqrt{35.0} = 32500 \text{ N/mm}^2 \]

Now

\[ \varepsilon_c = \frac{\sigma_c}{E_c} = \frac{\delta h}{h} \]

\[ \therefore \delta h = \frac{\sigma_c \cdot h}{E_c} \quad - \quad (i) \]

where \( \delta h \) = displacement or slip

\( \sigma_c = \) compressive stress of concrete under rib for a bond stress of 3.70 N/mm\(^2\) (cf. appendix 2)

\( h = \) height of concrete over which the compressive stress is dissipated (taken to be 5-7 times the mean rib height, cf. Rehm(33))

\[ \therefore \delta h = \frac{25.5 \times 3.5}{32500} = 0.00275\text{mm} \]

Actual slip measured for the no-stress specimens at \( \sigma_b = 3.70 \text{ N/mm}^2 \) is 0.0031mm which is of the same order of magnitude as the calculated value above.

At Elevated Temperatures

(a) E values for actual slip (no-stressed specimens)

Actual slips measured for no-stress specimens at \( \sigma_b = 3.70 \text{ N/mm}^2 \) are 0.0062 and 0.0148mm for 150 and 300°C
respectively.

If it is assumed, in equation (i), that the increase in temperature affects only the $E_c$ value then its percentage reduction can be calculated as follows.

At $150^\circ C$

percentage of $E_c$ value at $20^\circ C = \frac{0.0031}{0.0062} \times 100 = 50\%$

At $300^\circ C$

percentage of $E_c$ value at $20^\circ C = \frac{0.0031}{0.0148} \times 100 = 21\%$

Fig. 5 shows that the expected reduction is about 78 and 53\% respectively. The lower values of the calculated results could be due to differences in the type of concrete and test conditions. There is also the possibility that the height $h$ also changes slightly due to temperature. Elevated temperatures cause a greater bond length to be required to distribute the stresses adequately, therefore, it could be true also that a greater height of concrete is needed to dissipate the concrete compressive stress for increased temperature. If this is so for the measured slip and a constant stress an increase in $h$ would result in an increase in $E_c$ hence decreasing the magnitude of the percentage reduction.

(b) $E$ values for apparent slip (stressed specimens)

Apparent slips measured for stressed specimens (i.e. $\sigma_b = 3.70$ N/mm$^2$) after the heating cycle are 0.0725 and 0.15 mm for 150 and $300^\circ C$ respectively.

At $150^\circ C$

percentage of $E_c$ value at $20^\circ C = \frac{0.0031}{0.0725} \times 100 = 4.28\%$

At $300^\circ C$

percentage of $E_c$ value at $20^\circ C = \frac{0.0031}{0.15} \times 100 = 2.07\%$
This is a vastly reduced value compared with the results expected, especially when the stressed specimens could be expected to give a higher $E$ value than the no-stressed values, a point made in ref. 7.

Therefore it is concluded that the increase in the apparent slip of stressed specimens compared with the actual slip of the no-stress specimens is due to the effects of creep.