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The Behaviour of Steel Reinforcement in Composite Slabs at Elevated Temperatures

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Declaration

I declare that this thesis has been composed solely by myself, under the supervision of Professor Tim Stratford, and that it has not been submitted, in whole or in part, in any previous application for a degree or qualification. Except where stated otherwise by reference or acknowledgment, the work presented in this thesis is entirely my own.

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Abstract

This thesis investigates the potential rupture of modern deformed steel reinforcement around the boundaries of composite or reinforced concrete slabs with continuity over perimeter supporting beams as tensile membrane action develops in fire scenarios. The potential for rupture of modern deformed reinforcement in these conditions is not known from previous research but it is critical to the integrity of reinforced concrete and composite floor structures in fire scenarios. The primary aim of this research is to determine whether modern deformed reinforcement will be more prone to rupture than plain round reinforcement at hogging cracks which form around the perimeter of a slab during the development of tensile membrane action in a fire scenario.

Reinforcement bond has been experimentally investigated at elevated temperatures because bond is one of the main mechanisms which must be understood in order to understand the rupture of the reinforcement at crack locations. Pull-out tests have been conducted at temperatures up to approximately 600°C on plain round and deformed reinforcement of different diameters. This temperature range is expected to cover the range of bond temperatures which are realistic or applicable from a structural fire design perspective. The response of bond at elevated temperatures has been characterised in terms of stress and slip. A modified local bond stress-slip model which is applicable at elevated temperatures has been developed.

A finite element model has been developed to represent a region of reinforced concrete slab subject to flexural hogging with severe crack development. The purpose of the modelling is to investigate reinforcement behaviour and potential rupture when tensile membrane action develops at elevated temperatures. The

model incorporates the stress-slip bond response from the results of the pull-out tests.

It was found that slabs reinforced with deformed bars will perform similarly well to slabs reinforced with plain round bars in facilitating the cracking and rotation which occurs around the perimeter of a slab during the formation of tensile membrane action in a fire scenario. Increased slab depth was found to substantially reduce the ability of a slab to facilitate the cracking and rotation which occurs in this scenario. The effects of bar diameter and temperature were also investigated. It was found that the assumption of perfect bond, with no slip, was appropriately, and not excessively, conservative when modelling hogging reinforcement in this scenario. The implications of these research outcomes for reinforced concrete and composite slab design have been explained.

Lay Summary

Tensile membrane action is a mechanism which can provide additional strength to a reinforced concrete floor slab, but only when the slab has very large vertical deflections. This results in large cracks developing in the slab near to edges of the slab bay. In structural fire design, this mechanism is utilised to produce more efficient designs as the large deflections are generally acceptable in a fire scenario. For the mechanism of tensile membrane action to fully develop, the steel reinforcement bars must not break at the location of the large cracks in the concrete slab.

The current design methods utilising tensile membrane action are based on observations of full-scale tests where the older, smooth surfaced, type of reinforcement bars were used, and they did not break during the development of tensile membrane action. It is not known whether modern deformed reinforcing materials, which typically have a ribbed surface pattern and different material properties, will be more prone to breaking at these crack locations. The main aim of this research was to improve understanding of how modern reinforcing bars perform at the locations of these large cracks which develop during the formation of tensile membrane action in a fire scenario in order to determine the adequacy of current design methods with modern reinforcement. The effects of temperature, reinforcement bar diameter and slab depth on this scenario have also been considered as these are significant design considerations.

In order to understand the behaviour of the steel reinforcement in the situation of interest, it was necessary to investigate the bond interaction between modern deformed steel reinforcement and concrete at elevated temperatures. The elevated

temperature bond behaviour was characterised using experiments where reinforcing bars of different diameter and type were pulled out from concrete at different temperatures. The bond behaviour of smooth reinforcement bars and modern deformed reinforcement bars was characterised in terms of the stress and slip which occurs at the bond interface at different temperatures. These results were used to develop a modified form of a local bond model, which can be used to calculate bond stress and slip at elevated temperatures and will be useful in further numerical modelling based structural research.

A finite element model was developed to investigate the performance of modern deformed reinforcement at the location of cracks which develop near the edges of a concrete slab during the development of tensile membrane action in a fire scenario. The result of the bond experiments were used in modelling the interaction between the steel and concrete. The finite element model also simulated the development of a crack developing in the concrete slab due to bending.

It was found that modern deformed reinforcement performs comparably well to smooth reinforcement in the scenario being considered. Slab design methods utilising tensile membrane action do not need to be changed for use with deformed reinforcing bars in place of smooth bars, within the range of the parameters which were considered in this research. Bar diameter, slab depth and temperature were all found to be significant factors in the fire scenario being considered. The implications of the research outcomes for structural designers have been explained.

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1. Introduction

With large deflections, reinforced concrete and composite slab structures can benefit from increased load bearing capacity due to the development of tensile membrane action. As tensile membrane action develops, full depth cracks develop in the hogging region of the slab to facilitate rotation around the boundaries, as illustrated in figure 1.1. Figure 1.1 is an example arrangement, but tensile membrane action can also form with other boundary conditions, as described in section 2.3, and with composite slabs, such as that shown in figure 1.2. The increased load bearing capacity from tensile membrane action is utilised in modern performance-based structural fire design approaches. These methods are built on the observations of previous full-scale fire testing, referred to as the Cardington fire tests.

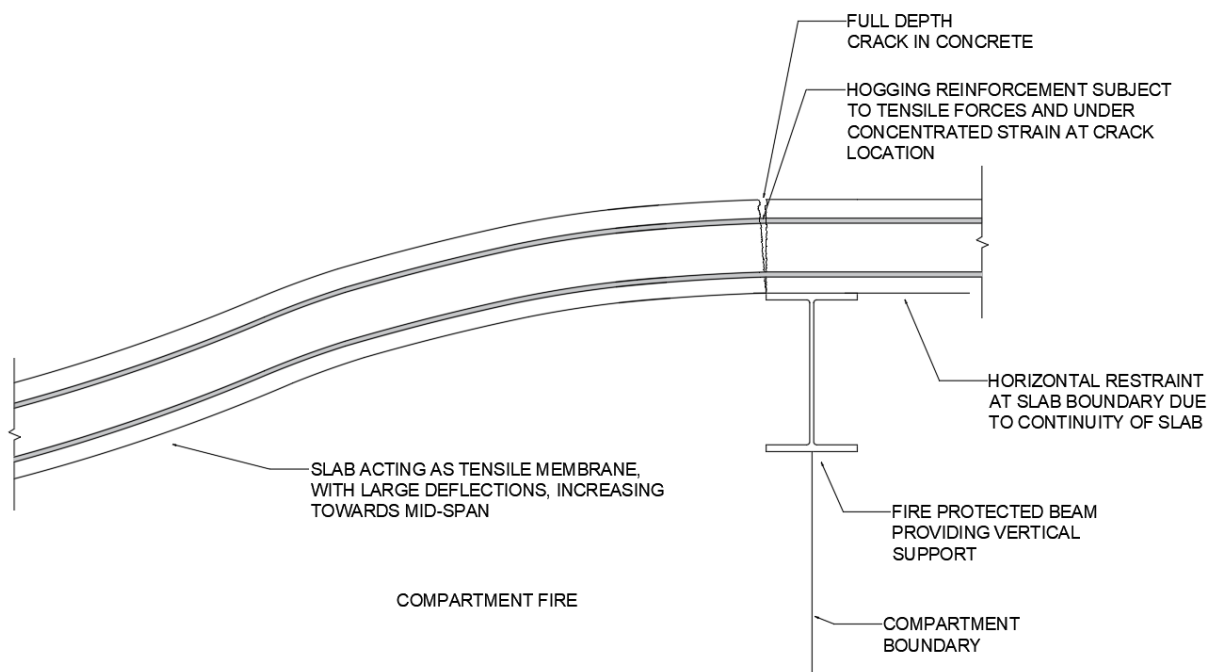


Figure 1.1. Indicative cross-section drawing of a boundary region of a reinforced concrete slab with full-depth cracking and concentrated strain of the hogging reinforcement

For tensile membrane action to fully develop, the hogging reinforcement of a slab must not rupture at the locations of the perimeter cracks. Although the slabs in the Cardington fire tests did not collapse, these tests utilised the older plain round type of reinforcement bar, rather than modern deformed reinforcement. It is unknown from previous research whether deformed reinforcement will be more prone to rupture than plain round bars in this scenario as different bond characteristics may increase the concentration of axial strain in the reinforcement at crack locations. It is important that the performance of deformed reinforcement in this scenario is understood as reinforcement rupture could result in structural collapse. This research investigates the behaviour and potential rupture of reinforcing bars around the boundaries of reinforced concrete and composite slabs as they develop tensile membrane action in a fire scenario.

The behaviour of reinforcing bars at crack locations in a fire scenario depends on the bond between the steel bar and the surrounding concrete at elevated temperature. The effects of elevated temperature on the bond between concrete and reinforcing steel, using modern materials, has also been investigated. This investigation has been conducted through experimental and numerical research, as described in this thesis.

1.1 Background

In 1996 the Building Research Establishment (BRE) conducted fire tests on an eight-storey composite steel-framed test-building at their Cardington Laboratory. The main aim of BRE's tests was to investigate the behaviour of a composite steel-frame building subjected to a series of compartment fires (Bailey *et al.*, 1999). In the Cardington fire tests, high vertical displacements of the composite floor slab were

observed, but no structural collapse occurred (Bailey, 2004). It was concluded that the composite floor slabs had acted under tensile membrane action.

Tensile membrane action can allow the capacity of composite slabs to exceed flexural capacity when under large vertical deflections (Bailey, 2003). Generally, the displacements required for tensile membrane action to occur would be unacceptable, but they are considered to be acceptable under fire, provided overall collapse or breach of compartmentation do not occur (Bailey, 2004; Girolardo, 2008).

Performance based design approaches which utilise the benefits of tensile membrane action have been developed (Bailey, 2004). Relative to traditional prescriptive approaches to structural fire design, performance based methods can bring benefits in cost-savings, construction time and reduction in building weight. The detailed response of reinforcement bars at the fire limit state is, however, not fully understood.

Modern reinforcement bar has a deformed shape which provides increased bond strength compared to the traditional plain round bar which was used in the Cardington tests. Significant hogging moments will be present in a reinforced concrete or composite floor slab if there is continuity over vertical support beams. These moments, in addition to tensile membrane forces will result in high levels of tension in the reinforcement in these areas (Flint *et al.*, 2013). If the stronger bond of deformed bars resulted in less slip of the bar relative to the surrounding concrete then it could be expected that this would result in greater concentration of strain within the reinforcement bars at crack locations. The implications of the use of deformed reinforcement bar in reinforced concrete and composite structures at the fire limit state are not well understood.

Some limited research has been conducted previously into the behaviour of steel reinforcement in concrete at elevated temperatures. The focus of the previous research has predominantly been on the effect of elevated temperatures on bond strength between steel and concrete at elevated temperatures. A few notable studies were conducted in the early 1980s (Diederichs and Schneider, 1982; Hertz, 1982; Morley and Royles, 1983); however, reinforcing materials have changed since this period. In the UK, the technical classification of deformed reinforcement has become more specific, with the inclusion of detailed requirements of rib height, inclination angle, and increased permitted rib spacing (British Standards Institution (BSI), 1978; BSI, 2005). The required strength of reinforcing steel has also increased in modern standards. Modern reinforcing steel in the UK also comes in different ductility classifications, each with lower specified ductility requirements than older plain round bar. Additionally, the use of deformed reinforcement has become increasingly widespread.

Changes in bond response of modern deformed reinforcement could increase the localisation of axial strain in reinforcement at crack locations and, potentially, make the reinforcement more prone to rupture. Additionally, changes in mechanical properties of reinforcement will change the response of the reinforcement in this scenario. It is important, therefore, to investigate whether the assumptions behind tensile membrane action in fire are still valid with modern deformed reinforcement.

This research is relevant to both composite slabs and reinforced concrete slabs which have continuity over perimeter supporting beams. Composite slabs are a commonly used form of structure which use a thin steel profiled deck as a permanent formwork that also provides external reinforcement. Figure 1.2 shows a typical composite slab arrangement with a steel beam below. Additional hogging

reinforcement will often be included in the upper part of the slab in the hogging region. The slabs are connected to downstand steel beams by welded shear studs at regular intervals along the span of the beam. The term "composite slab" refers to the slab spanning between adjacent downstand steel beams. The term "composite beam" refers to the overall slab/beam arrangement in the direction of the steel beam span (Johnson, 2004). Historical fires and large-scale fire tests have demonstrated that prescriptive design methods can severely underestimate the fire resistance of composite slab and beam structures.

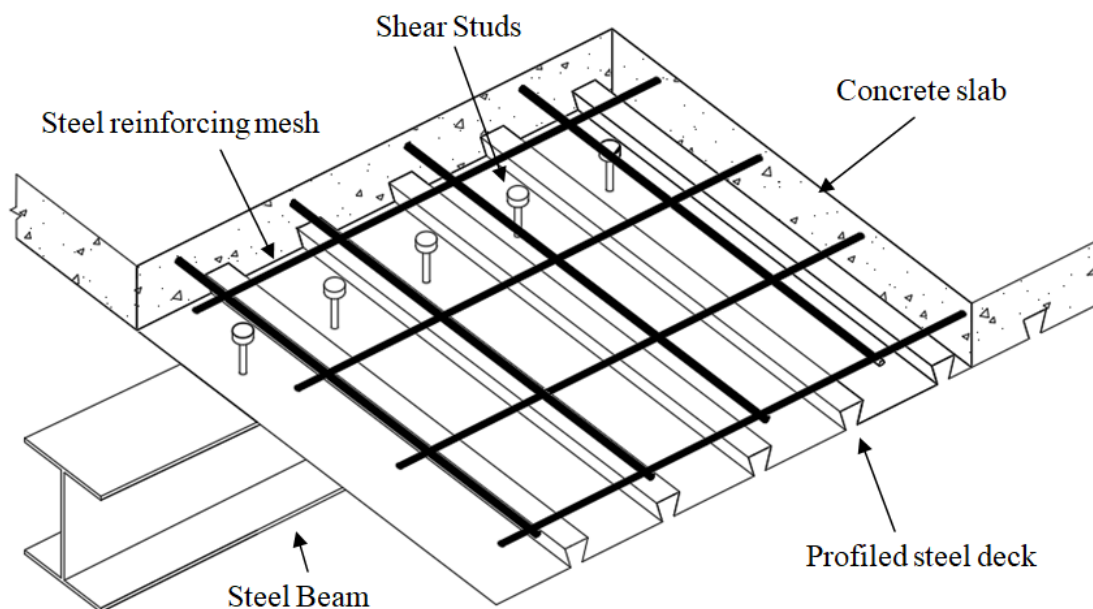


Figure 1.2. Cutaway drawing of a typical composite slab

1.2 Research aims

The aim of this research is to determine whether modern deformed reinforcement will be more prone to rupture than plain round reinforcement at hogging cracks which form around the perimeter of a slab during the development of tensile membrane action in a fire scenario. The reason for investigating this behaviour is that it is not known from previous research whether the differences in surface geometry and

ductility of modern deformed bars will make them more susceptible to reinforcement rupture. This is necessary in order to determine whether current tensile membrane action design methods are adequate or if there is considerable risk of reinforcement rupture in slabs reinforced with modern deformed bars.

In order to meet the research aim, the objectives of this research are:

- characterise the elevated temperature bond behaviour, in terms of bond stress, and slip of plain round and modern deformed reinforcing steel. This information is required in order to meet the aim because bond slip will substantially influence the localisation and rupture of the reinforcement.
- evaluate the performance of modern deformed reinforcement, relative to plain round reinforcement, in terms of susceptibility to rupture at crack locations around the perimeter of a slab during the development of tensile membrane action in a fire scenario.
- determine the effect of temperature, bar diameter and slab depth on the ability of slabs to develop tensile membrane action without reinforcement rupture. This is to be investigated as these are significant design parameters with the potential to affect the response of the crack-spanning reinforcement.

1.3 Thesis outline

This thesis contains eight chapters plus appendices. Chapter 2 provides a literature review explaining relevant concepts in structural fire design. This includes an overview of the relevant observations from the Cardington fire tests and the research into tensile membrane action which followed. There is also an explanation of the previous research and current knowledge of reinforcement bond behaviour at elevated temperature.

Chapter 3 describes an experimental programme investigating bond behaviour at elevated temperature. A proposed modified local bond stress-slip model is described in chapter 4. Chapter 5 explains the numerical modelling methodology used in this research. Chapter 6 describes the numerical modelling simulations which were conducted to investigate the response of crack-spanning reinforcement at elevated temperatures. Chapter 7 documents the results, analysis and discussion of the numerical modelling research. Chapter 8 presents the conclusions from this research along with suggestions for further work in this area of research.

2 Literature review

This chapter provides a review of relevant literature and explains the current knowledge which is of relevance to this research. The chapter provides an overview of research into elevated temperature bond behaviour and tensile membrane action. Section 2.1 introduces some concepts of structural fire design which are fundamental to the rest of this literature review and the current research. Section 2.2 describes the relevant behaviours which have been observed in previous building fires and full-scale testing.

Section 2.3 explains tensile membrane action behaviour in a fire scenario. Bond behaviour at ambient temperature is explained in section 2.4 as it is important to understand ambient behaviour before investigating bond behaviour at elevated temperatures. Section 2.5 describes some of the more common forms of bond test method and the advantages and disadvantages of each method. The relevant effects of elevated temperature on concrete and steel are explained in section 2.6. Section 2.7 gives an overview of the concrete material model which has been used in the numerical research which is presented in chapter 5. Section 2.8 explains the current knowledge of bond behaviour at elevated temperatures.

2.1 Structural fire design

It is necessary to have an understanding of some of the central concepts underpinning structural fire design methods in order to be able to evaluate the appropriateness of the current design assumptions and the implications of the current research on design. Structural fire design requires that the fire resistance of a structure is greater than, or equal to, the fire severity. This condition can be verified in terms of either time, temperature or strength. The most common procedure is to quantify both fire resistance and fire severity in terms of time (Buchanan, 2017).

2.1.1 Prescriptive and performance based design

Design for fire safety has traditionally followed a prescriptive approach in which designers follow prescriptive building design codes (Buchanan, 2017). In recent decades, there has been a shift towards the adoption of performance based building design codes. A performance based design approach allows designers to produce more comprehensive fire engineering designs based on first principles. This grants more freedom to produce innovative solutions, provided the adequacy of the design can be demonstrated to the requirements of the relevant authorities.

Designers following a prescriptive approach may 'blindly' apply fire protective materials to any exposed steel without considering the actual response of the structural assemblies to fire. Following a performance-based approach, however, the designer may take account of the expected behaviour, potentially justifying the use of substantially less fire protection materials (Bailey, 2004). This can result in significant savings in cost and construction time.

Dissatisfaction with the prescriptive approach to fire safety engineering is not due to the poor outcomes in terms of safety, as fire deaths in the UK are at lower levels

than in living memory (Spinardi, 2016). The dissatisfaction with prescriptive design in fire safety engineering is due to the limiting effect on innovation and what are considered to be overly restrictive and costly requirements.

2.1.2 Fire severity

The concept of fire severity is to provide a quantification of the potential of a fire to cause destruction. Fire severity is generally expressed in terms of a time period of exposure to a standard fire specified by a building code, or an equivalent time calculated. Building elements are typically given fire resistance ratings derived from fire tests or calculation.

Prescriptive design codes generally dictate a specific standard fire profile. Design to performance based codes generally uses a fire severity of a complete burnout fire or exposure to a standard fire for a time which would cause an equivalent impact as a complete burnout (Buchanan, 2017). In some instances the design fire may be reduced and intended only to allow fire fighting response or escape.

2.1.3 Fire resistance

Fire safety design objectives are commonly achieved through a combination of passive and active fire protection. Active fire protection includes systems and actions to control fire such as, for example, water sprinkler systems. Passive systems, however, are built into a building and do not require any specific action to be taken at the time of a fire.

Fire resistance is the most important part of passive fire protection and it is a fundamental concept in structural fire design. The fire resistance of a structure, or structural element, is a measure of its ability to avoid structural collapse and prevent fire spread when subjected to a fire of a specified severity (Buchanan, 2017).

Modern design codes, such as BS EN 1991-1-2:2002 (BSI, 2002), do not require a structure to be undamaged after fire.

Time has traditionally been the favoured variable to quantify fire resistance. The fire resistance of a structural element is deemed acceptable if the time to failure is greater than the required fire duration (Buchanan, 2017). An alternative approach to confirm adequate fire resistance is to determine whether the temperature of a structural element will be maintained below a specific limit. The strength of a structural element can be assessed to ensure adequate load carrying capacity relative to the load applied during fire (Meacham *et al.*, 2005).

2.1.4 Standard and real fires

The standard fire test has widespread international recognition and is provided in a range of international standards for fire testing including BS 476-20:1987 (BSI, 2014), ISO 834 (ISO, 2014) and BS EN 1363-1 (BSI, 2020). The standard fire curve, shown in figure 2.1, is defined by the following relationship:

$$T = 345 \cdot \text{Log}(8 \cdot t + 1) + 20 \quad (2.1)$$

where:

T is temperature in degrees Celsius ($^{\circ}\text{C}$) at a time, t , measured in minutes.

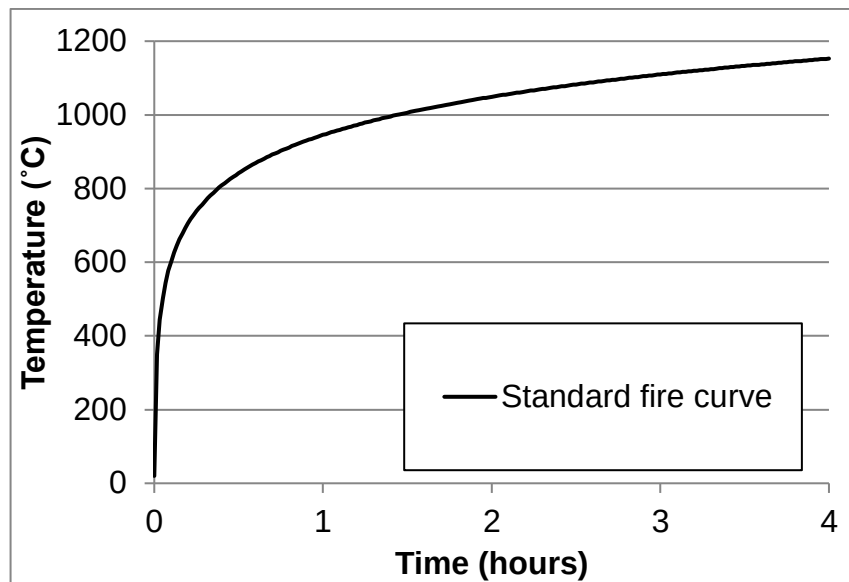


Figure 2.1. Standard fire curve from ISO 834

Throughout the test duration, the temperature is controlled by controlling the rate of the fuel supply to the test furnace. Standard fire tests measure fire resistance of structural elements in terms of integrity, load bearing capacity and insulation.

Integrity of an element is retained when hot gases or flames are unable to pass through cracks, although other limitations must be considered (Buchanan, 2017). Insulation must limit the temperature rise on unexposed surfaces to specified levels.

The standard fire test has a number of limitations. One criticism is that it does not represent real natural fire conditions. In a standard test, the temperature simply increases continuously over time. However, in a natural fire, the temperature would increase to a maximum, before decreasing again during the decay phase (Buchanan, 2017).

A further disadvantage of the standard fire test is that it is limited to testing only individual elements or connections. Standard fire tests cannot be used to study the behaviour of structural assemblies which would feature interaction between sets of

elements and boundary conditions. This limitation is significant as restraint can greatly influence the structural response of a structural element under fire conditions (Usmani *et al.*, 2001). This is relevant to the current research as composite slab behaviour, particularly tensile membrane action, is dependent on the interaction of the slab with the surrounding structure.

Prescriptive methods for structural fire safety design are based upon the results of such standard fire resistance tests for individual structural elements. Therefore, they do not account for the interactions between structural elements which occur due to thermal stresses under fire conditions. Consequently, prescriptive methods are unable to quantify the actual level of fire safety in any building.

2.1.5 Heat transfer in reinforced concrete and composite slabs

The load bearing capacity of elements subjected to fire depends on the internal temperatures. In a reinforced concrete or composite slab, temperature gradients develop as the slab is heated, with the concrete near the surface which is exposed to the heat source heating more rapidly while the concrete further from the exposed surface, and usually the reinforcement, will heat more slowly due to the inherent insulating properties of concrete. Heat transfer modelling is commonly used in structural fire engineering as part of the determination of fire resistance. The temperature gradients within an element depend on the conduction of heat within the element and the convection and radiation of heat at the surfaces and any voids.

Heat transfer models require definition of element geometry, heat transfer coefficients at boundaries and the thermal properties of the materials. One of the complicating aspects of heat transfer modelling is the temperature dependency of material thermal properties. Finite element models can be developed to estimate the

changing temperature distribution in an element over time and these methods are useful in both industrial and research applications.

Heat transfer is typically considered as either in a steady-state condition or a transient condition. In real fires, temperature is transient, but many structural fire experiments are conducted with quasi-steady-state temperatures, where temperatures change slowly enough that they are considered to be effectively constant. The properties of conduction, convection, radiation, specific heat capacity and density were all considered within the elevated temperature numerical modelling work described in chapters 6 and 7, as described in section 6.5.

2.1.5.1 Conduction

Thermal conduction is the diffusion of heat within a body, or bodies in contact. Thermal energy is conducted within a body from hotter zones to cooler zones. Fourier law connects linearly the heat flux density to the temperature gradient (Bergheau and Fortunier, 2013). Heat transfer by conduction within an element is governed by Fourier Law, which, for a one dimensional case, has the form:

$$q = -k \frac{dT}{dx} \quad (2.2)$$

where the term q represents heat flux, k is thermal conductivity, x is distance and T is temperature. The negative term indicates that the direction of heat flux is the direction of decreasing temperature.

Conductivity may be defined as isotropic, orthotropic or anisotropic. The temperature dependent thermal conductivity of steel, up to 600°C, according to Eurocode 4 Part 1-2 (BSI, 2005b) is shown in figure 2.2. The temperature dependent thermal

conductivity of concrete according to Eurocode 2 Part 1-2 (BSI, 2019) is shown in figure 2.3.

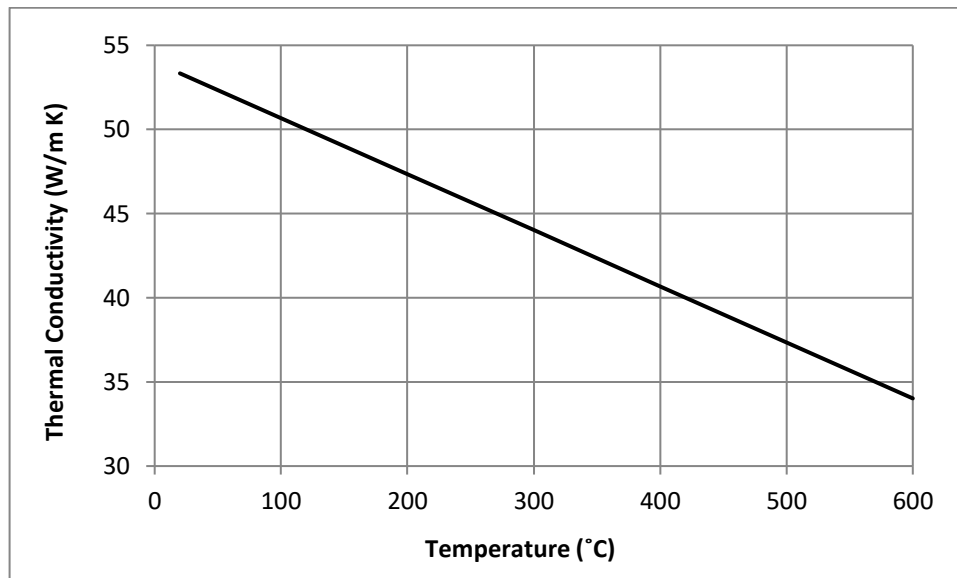


Figure 2.2 Thermal conductivity of steel from Eurocode 4 Part 1-2 (BSI, 2005b)

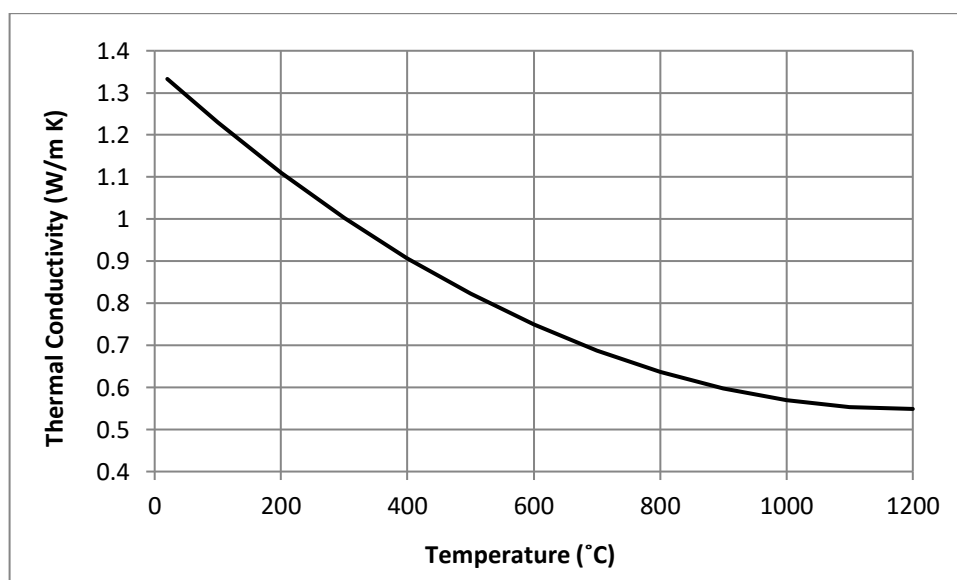


Figure 2.3. Thermal conductivity of concrete from Eurocode 2 Part 1-2 (BSI, 2019)

2.1.5.2 Convection

Convection is a form of heat transfer due to fluid movement. Convection is a significant factor in flame spread and the transport of smoke and hot gases during fire. Although convective heat transfer is always present in fire, it becomes more important relative to radiation as temperatures decrease. Heat transfer is typically taken to be directly proportional to the difference in temperature between two materials, where convective heat flow per unit area, q , is given by:

$$q = -h_c \Delta T \quad (2.3)$$

where h_c is the convective heat transfer coefficient and ΔT is the temperature difference between the fluid and solid surfaces.

2.1.5.3 Radiation

Radiation is the emission or transmission of energy by electromagnetic waves. Thermal radiation is an important mechanism and is often the dominant form of heat transfer between fire and solid surfaces of compartments. At a given point on the receiving surface, the radiant heat flux, q , is given by:

$$q = \phi \epsilon_m \epsilon_f \sigma (T_r^4 - T_m^4) \quad (2.4)$$

where ϵ_m is the emissivity of the receiving surface and ϵ_f is the emissivity of the fire. The configuration factor, ϕ , is a measure of the extent to which the emitting surface is 'seen' by the receiving surface. The term T_r is the effective radiation temperature of the fire environment. The term T_m is the surface temperature of the element (Buchanan, 2017).

2.1.5.4 Specific heat

Specific heat is the quantity of heat necessary to heat each unit mass of a material by one degree, and has the units J/kg K. The temperature dependent values of specific heat capacity for steel according to Eurocode 4 Part 1-2 are shown in figure 2.4 (BSI, 2005b). The spike in specific heat capacity of steel is due to a phase change where atoms transition from a face centred cubic structure to a body centred cubic structure (Kodur et al., 2010). Figure 2.5 shows the temperature dependent specific heat capacity of siliceous concrete with different moisture content, u , values according to Eurocode 2 Part 1-2 (BSI, 2019). These are design values which can be taken when moisture content is not considered explicitly in the calculation method.

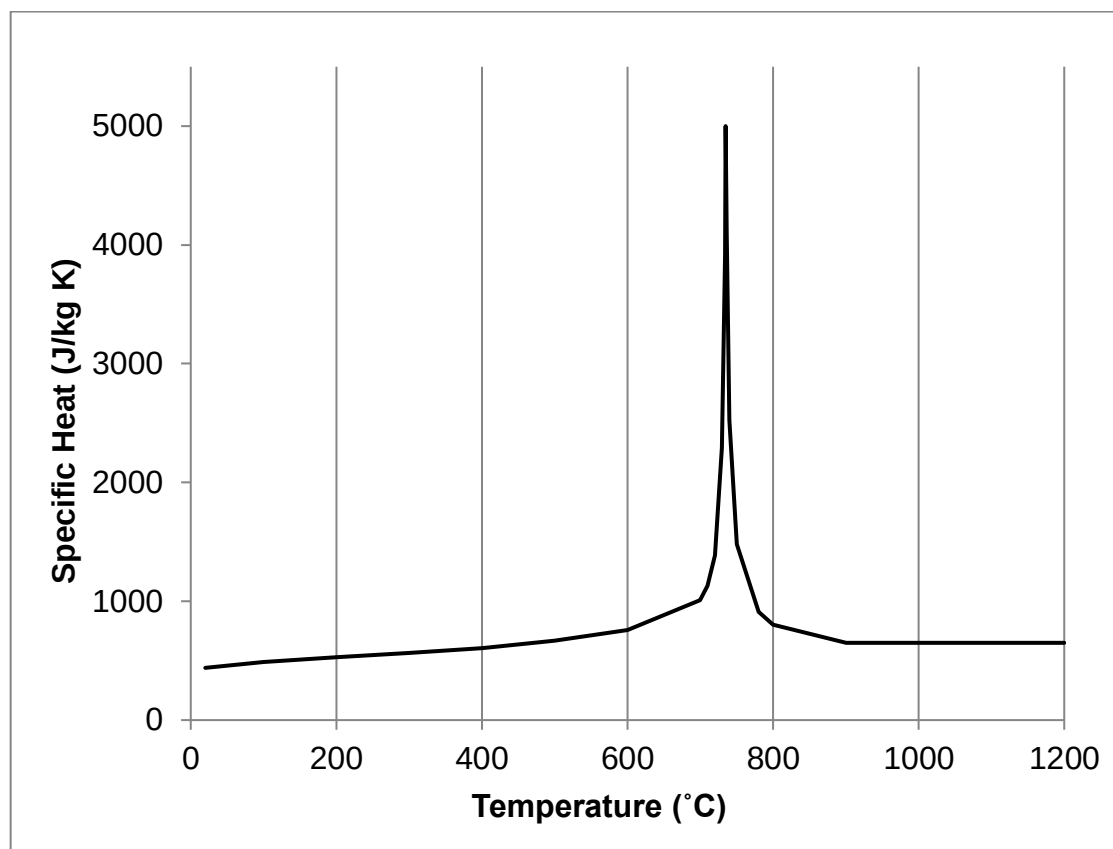


Figure 2.4. Temperature dependent specific heat capacity of steel from Eurocode 4 Part 1-2 (BSI, 2005b)

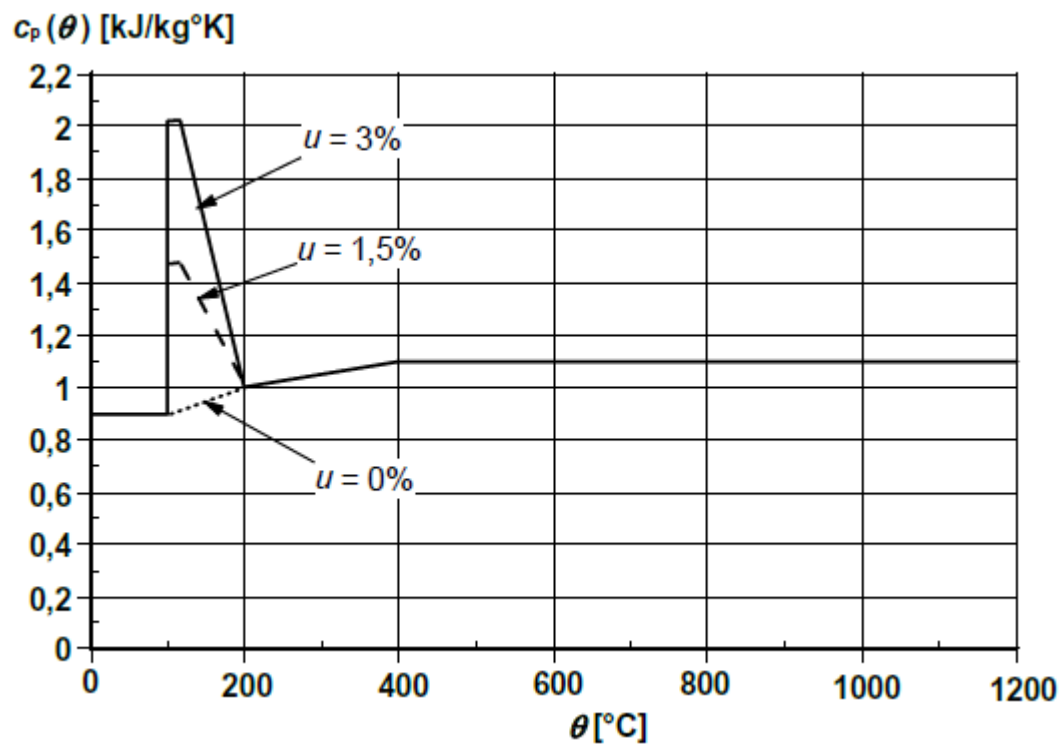


Figure 2.5. Temperature dependent specific heat capacity of siliceous concrete with different moisture content, u , values (BSI, 2019)

2.1.5.5 Density

Mass density is another parameter which is required in the calculation of heat transfer within a solid material. As per the recommendation of Eurocode 4 Part 1-2 (BSI, 2005b), the density of steel can be taken as a temperature independent value of 7850kg/m^3 . Temperature dependent values of mass density for concrete are given in Eurocode 2 Part 1-2 (BSI, 2019), as shown in figure 2.6 below.

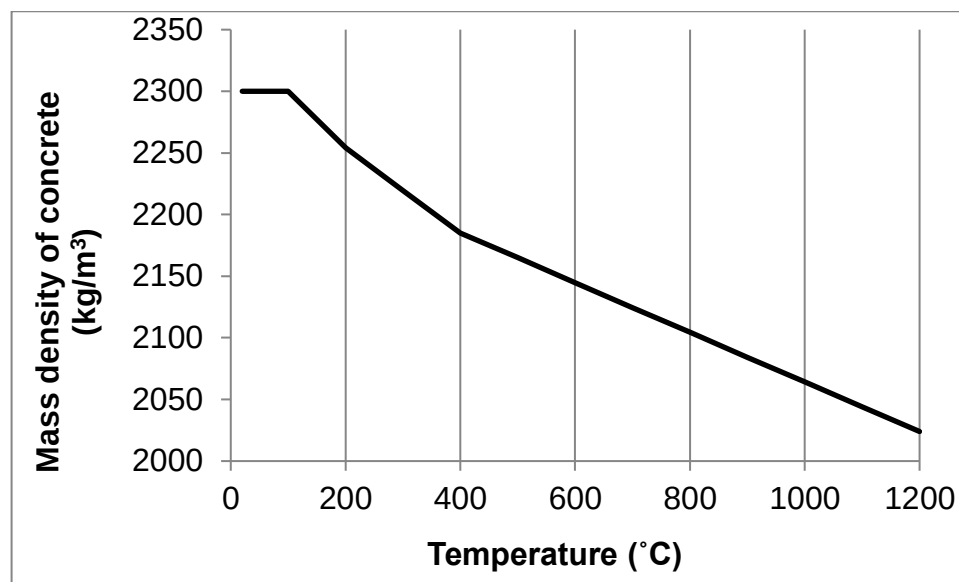


Figure 2.6. Temperature dependent mass density of concrete from Eurocode 2 Part 1-2 (BSI, 2019)

2.2 Behaviour of composite structures in fire

2.2.1 The Broadgate fire

A fire in the London based Broadgate development in 1990 changed perceptions in structural fire engineering. The Broadgate development was an office building with typical steel frame and composite floor slab construction. A fire broke out in the first floor of the building during construction in 1990. Although the building had been designed for a 90 minute fire resistance, there were still large areas of unprotected steel and the active fire protection and sprinkler systems were not yet operational. However, the structure was subjected only to dead loads at the time of the fire.

The duration of the fire was four and a half hours, with temperatures reaching over 1000°C for a period of two hours. There had been sizable deflections, including the shortening of columns by up to 100mm and the permanent deflection of the floor by 600mm in some areas. Despite these observations, there was no failure of the structure, and the integrity of the floor slab was maintained (Bailey *et al.*, 1999; Torero and Lane, 2004).

The structure had not behaved in a manner which could be represented by standard furnace testing of individual elements. It became more apparent to engineers that the approach of assessing structures based on the performance of isolated elements was over-conservative and unscientific (Usmani *et al.*, 2001) The observations from Broadgate prompted further research into composite structures in fire.

2.2.2 The Cardington fire tests

Between 1995 and 1997, a major programme of full-scale fire tests was conducted within the BRE facility at Cardington, England. The Cardington tests were substantially influenced by the observations from the Broadgate fire. The outcomes of these tests and the subsequent research work are the foundations of this research and they have been described here in order to put the current research in context. The content of this section is based on the descriptions of these tests given in Bailey (1999), British Steel (1999) and O'Conner *et al.* (2003).

2.2.2.1 Description of tests

The fire tests, referred to as the Cardington fire tests, were conducted on an eight - storey framed building. The structure, shown in figure 2.7, had a traditional steel and concrete composite construction which is typical for a city-centre eight-storey office. The structure was designed in accordance with the relevant British Standard structural design code and the design was also checked for compliance with Eurocodes.



Figure 2.7. Cardington test structure (British Steel,1999)

2.2.2.2 Aims of test

The main aim of the test programme was to investigate the actual response of this type of structure when subjected to localised compartment fires. The behaviour of the building was observed under test conditions. The structure was tested in an attempt to identify the inherent strength of the structure in fire allowing evaluation of the design factor of safety. The results of the tests were intended to be utilised in the development and validation of computer models for further investigation of structural fire scenarios. The findings were also useful in the development of design guidance.

The design was intended to be relatively simple, with minimised material use. The construction process was intended to be representative of what was normal building practice in the UK at that time.

2.2.2.3 The test structure

The building had a height of 33m and measured 45x21m on plan. There were five bays of equal span along the length and three bays of 6m, 8m and 7m span across the width of the building, as shown in figure 2.8. The building featured a central lift core and two stairwells, located at opposite ends of the structure.

The steel frame featured lateral restraint in the form of cross-bracing. On each floor level, a composite floor slab was supported by standard UB steel beams. The floor slab was 130mm deep and was formed of a standard steel trapezoidal deck and contained steel reinforcing mesh formed of 6mm diameter bars welded at 200mm spacing. This slab was quite shallow and the bars were of small diameter by modern standards, in addition to being the older plain round style of reinforcing bars. Each floor of the structure had a similar layout.

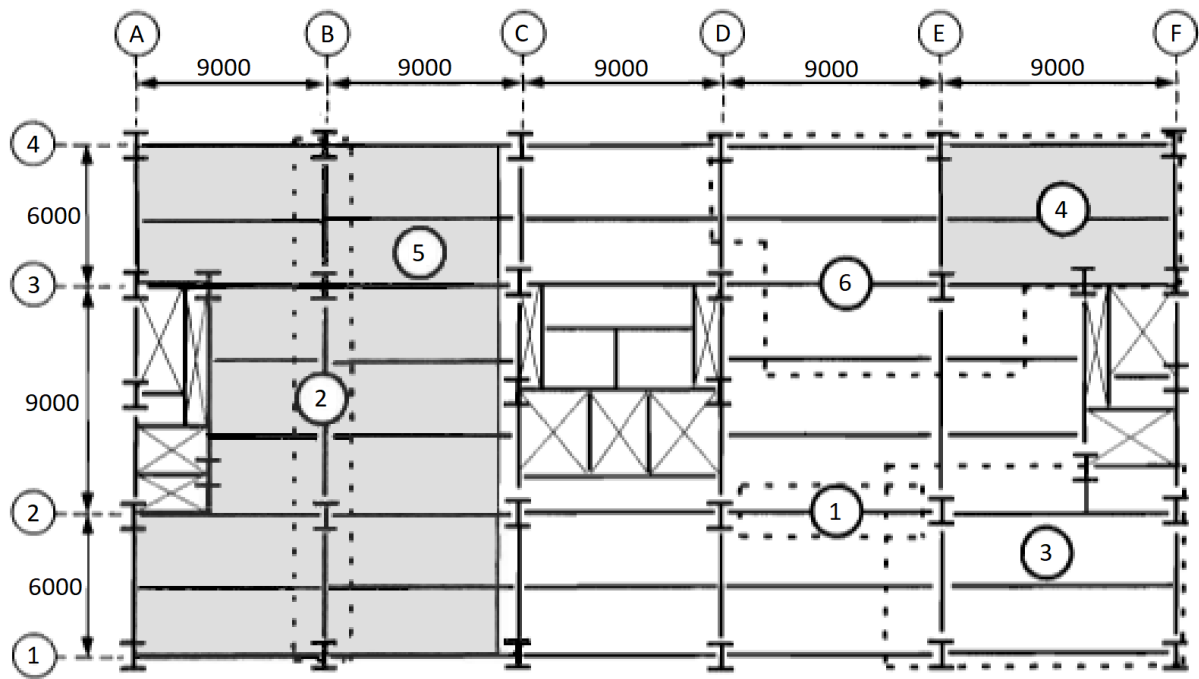


Figure 2.8. Typical plan view showing locations of fire tests with BRE test locations shaded, and with dimensions in millimetres, modified from Bailey et al. (1999).

2.2.2.4 Loading

Sandbags were used to achieve an overall load of 5.48kN/m^2 on each floor, including structural self weight. The sandbags, 11kN each, were placed at uniform intervals across the floors representing a relatively uniformly distributed load.

2.2.2.5 The fire tests

Six localised fire tests were conducted at different locations within the test structure. The tests differed mainly in terms of the overall fire compartment size, the rate of fire growth and levels of restraint provided to beams. Four of the six tests were conducted by British Steel and two tests were conducted by the Building Research Establishment (BRE). The description in the following sections focuses on the BRE tests. The tests are relevant to this research as they provided a clear demonstration

of tensile membrane action in a fire scenario and they are the basis for current design assumptions of tensile membrane action.

2.2.2.6 Corner test

One of the BRE tests featured a 9m x 6m compartment which had been constructed in one corner of the building at the second floor. The location of the test is marked as '4' in figure 2.8. The perimeter of the compartment was formed by fire resistant board on stud partitions and the columns were provided with fire protection. The steel beams and composite slab were left unprotected. The compartment featured windows and doors which were kept closed and no ventilation was provided, but the compartment was not artificially sealed. It was assumed that the window glass would crack in the early stages of the fire but this did not occur in this test and the fire brigade removed some of the glass during the test to increase ventilation.

A fire load of 40kg/m² was provided by timber cribs. The highest recorded gas temperature at the centre of the compartment was 1051°C, measured 102 minutes after ignition. The highest steel temperature was 903°C after 114 minutes. The design codes had suggested the beam would 'fail' at a temperature of 680°C (Bailey *et al.*, 1999).

Despite these temperatures, there was no collapse of the structure. The maximum displacement of the composite floor was 269mm at the centre of the compartment. This displacement is more than twice the depth of the floor slab (Bailey *et al.*, 1999).

2.2.2.7 Large compartment test

The other BRE test was on a larger compartment, with a floor area of 340m². The location of the test is marked as '5' in figure 2.8. Similarly with the corner test, the compartment also had fire resistant partitions at the perimeter and the columns were

fire protected. The composite floor was, again, unprotected. Unlike the corner test, some of the windows were left open to allow ventilation (Bailey 1999).

The fire load and fuel were the same as in the corner test. The highest recorded gas temperature was 763°C and the highest steel temperature was 691°C. The fire scenario was fairly severe as the fire remained at high temperatures for a longer duration (Bailey *et al.*, 1999).

Despite these temperatures there was no collapse of the structure. There was some local buckling in the internal downstand beams. The largest recorded displacement of the composite floor was 557mm, which is more than four times the depth of the slab (Bailey *et al.*, 1999).

2.2.2.8 Relevant observations from Cardington

The highest steel temperatures were 903°C and 1100°C in the BRE and British Steel tests respectively yet there was no sign of collapse in any of the six tests. This demonstrated that the design code calculation, that the beam would fail at 680°C, was highly conservative. It was recognised that the primary reason for the superior performance of the structure under fire was the contribution of the composite floor. The composite floor performed well in all six fire tests and demonstrated good fire resistance (Bailey *et al.*, 1999; O'Conner, 2003).



Figure 2.9. Internal view of British Steel test 6 showing large deflections of the floor above (British Steel, 1999)

As temperature in the beam increased, the beams lost much of their strength and the load was supported by the slab. At such large deflections, the load carrying mechanism was tensile membrane action, with tension being carried in the reinforcing mesh. As temperatures increased, the slab transitioned from a flexural state into tensile membrane action (Bailey *et al*, 1999; O'Conner, 2003). The large deflection is apparent in figure 2.9, a post-fire view inside the compartment of one of the tests by British Steel.

There was evidence of some concrete crushing near the compartment boundaries. From above the compartment, it was evident that the top of the concrete slab had cracked and that these cracks generally followed the path of the primary and secondary beams below, as can be seen from figure 2.10, a view of the slab from above the fire compartment after one of the British Steel tests. This cracking behaviour around the support lines is a feature of tensile membrane action (Bailey *et al.*, 1999; O'Conner, 2003). It is the behaviour of reinforcement at these cracked regions around the slab perimeter, as shown in figure 2.10, which is under investigation in this thesis.



Figure 2.10. Internal view showing cracking of the composite slab over support line from British Steel test 6 (British Steel, 1999)

2.3 Tensile membrane action

Real fires and tests such as the Broadgate fire and the Cardington fire tests have demonstrated that the fire resistance of composite framed buildings is enhanced by the composite behaviour of the structural elements and the mechanism of tensile membrane action. Reinforced concrete flat slabs can also benefit from tensile membrane action (Jiang et al., 2022). Typically, reinforced concrete and composite floor slabs act in flexure. The yield line theory is commonly used to determine the flexural strength of slabs with small deflections (Wood, 1961). In this approach, the flexural capacity of the cross section at the yield line is considered.

If deflections increase, the loads may be resisted through a mechanism known as compressive membrane action (Zheng et al., 2008). Compressive membrane action is an arching action which can develop in slabs which have lateral restraint, and it requires that the centre of the compressive forces is lower at the supports than at the mid-span of the slab. Compressive membrane action can only develop in slabs where the vertical deflection does not exceed the depth of the slab.

As deflections increase further, the compressive membrane forces no longer act and the slab is supported by tensile membrane action. In tensile membrane action, the tensile forces are resisted by the steel reinforcement, which acts as a tensile net. In slabs which are horizontally restrained, the tensile membrane is supported by catenary action at the perimeter of the slab bay. Figure 2.11 demonstrates tensile membrane action in a slab which is horizontally restrained. Tensile membrane action can also develop in slabs which are not horizontally restrained, but in this case a compressive ring must form around the outer region of the slab, encircling the tensile region. Figure 2.12 illustrates the tensile membrane mechanism in a slab without

horizontal restraint. The compressive ring will not form in a slab which is horizontally restrained at the edges. Tensile membrane action is a stronger mechanism in slabs with horizontal restraint because the full slab is able to act as a tensile membrane with the benefit of catenary action.

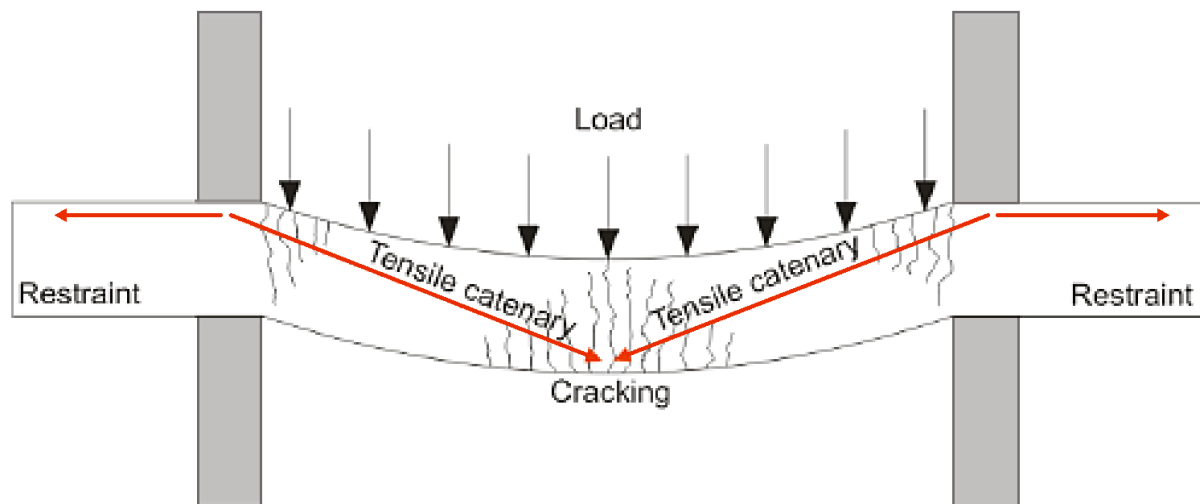


Figure 2.11. Cross section illustration showing the principle of tensile membrane action with horizontal restraint at boundaries, modified from Botte *et al.* (2014)

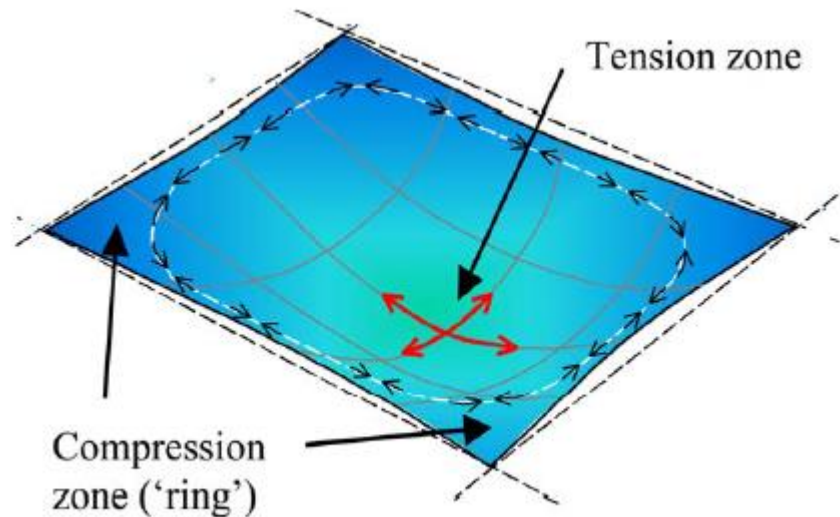


Figure 2.12. Tensile membrane action of a floor slab without horizontal restraint around the perimeter (Bailey, 2004)

Tensile membrane action of reinforced concrete slabs was the subject of research since, at least, the 1960s (Park, 1964). Research remained relatively limited as the deflection which is required to induce tensile membrane action exceeds the serviceability limits of typical conditions under normal working loads. In recent decades, tensile membrane action has received greater research interest due to the recognition that it can enhance the performance of reinforced concrete structures in fire. The reason for its applicability in structural fire engineering is that the increased deflections which are required for tensile membrane action to develop are considered acceptable within the fire limit state.

In structural fire design, tensile membrane action may be utilised provided that the deformation of the floor slab does not permit the fire to penetrate the boundaries of the fire compartment. By taking advantage of tensile membrane action in design, it is possible to create more efficient designs with cost savings from reduced use of fire

protection (Bailey, 2003). However, in order to utilise tensile membrane action in design, the behaviours and limitations of the mechanism need to be understood.

Tensile membrane action can be present in either flat concrete slabs or composite slabs. In a fire scenario that causes tensile membrane action, the steel decking and concrete ribs of composite slabs will reach such high temperatures that their stiffness and strength become negligible (Bailey et al., 2000; Jiang et al., 2022). The steel deck was observed to debond from the slab in the Cardington fire tests (British Steel, 1999). It is therefore, generally reasonable to simplify a composite slab as being equivalent to a flat slab with a thickness of the upper part of the composite slab in a fire scenario (Jiang et al., 2022). There will, however, be differences in thermal gradients with differences in the cross sectional shape.

2.3.1 Further Experimental Research in Tensile Membrane Action in Fire

Following the Cardington Fire tests, several researchers have conducted fire experiments on composite and flat concrete slabs in order to further study the structural behaviours (Bailey et al., 2000; Bailey and Toh, 2007; Dong and Zhu, 2011; Foster et al., 2004; Lim et al., 2004; Li et al., 2017). These experiments were carried out on simply supported slabs, without restraint against rotation or horizontal movement at the edges. These experiments demonstrate a failure by rupture of the reinforcement at mid-span of the slab when tested at elevated temperature.

Nguyen et al. (2015) tested three scaled composite slabs with different boundary conditions in fire conditions. The effect of rotational restraint was investigated in these tests. These slabs were also numerically modelled in Abaqus to further investigate the structural response of the slabs. Composite slabs may have rotational

restraint due to continuity over primary support beams. The slabs tested also differed in terms of the presence of unprotected interior secondary beams. The tests demonstrated that both the continuity of reinforcement over supporting beams and the presence of interior beams enhanced the load bearing capacity of the slabs. Interior beams significantly influenced the magnitude and distribution of the stress in the reinforcement. The results demonstrated that the presence of internal secondary beams may change the failure mode from that of reinforcement fracture at the mid-span to reinforcement fracture over the protected edge beams. This could not have been observed in previous experiments on slabs which lacked horizontal or rotational restraint, and it demonstrates the importance of considering the failure mode of reinforcement fracture over the edge beams. A limitation of the study by Nguyen (2015) is that the slabs were small-scale, and further testing should be conducted at full-scale to verify these findings.

2.3.2 Numerical Modelling of Tensile Membrane Action in Fire

Prompted by the experimental studies, extensive computer modelling has been conducted to simulate the development of tensile membrane action in slabs in fire scenarios (Franssen, 2005; Izzuddin et al., 2004; Jiang and Usmani, 2013; Kodur et al., 2013; Lim et al., 2004). Numerical modelling has generally considered slabs without horizontal or rotational restraint. The assumption of no horizontal restraint of slabs is based on an assumption that reinforcement will rupture at the edges due to large hogging moments and the membrane tension. Although this assumption is a conservative one, it does limit the extent to which the beneficial effect of tensile membrane action can be utilised in design due to the lack of consideration of catenary action.

Jiang and Li (2018) modelled slabs at elevated temperature, and investigated the significance of horizontal restraint. They found that without horizontal restraint, the failure of the slab was by rupture of the reinforcement at the slab centre for slabs with smaller aspect ratios, and at the intersection of yield lines for slabs of greater aspect ratios. Jiang and Li also modelled horizontally restrained slabs, and found that failure of the slab was due to reinforcement rupture along the long edges of the slab in this condition.

None of the previous numerical models for slabs in fire conditions account for the effect of bond slip between the reinforcing steel and surrounding concrete. As bond slip may alleviate the extent of strain localisation in the bar at crack locations, the failure of the slabs by reinforcement rupture may, therefore, not be accurately represented.

2.3.3 Tensile Membrane Action Design Methods

Analytical calculation methods considering the additional load bearing capacity from tensile membrane action have been developed for design purposes. Bailey and Moore (2000) proposed a method based on yield line theory which assumes no horizontal restraint and that failure is at the mid-span. As the assumption of no horizontal restraint limits the potential benefit of tensile membrane action, further design methods have been developed which consider slabs with horizontal restraint (Cameron and Usmani, 2005; Omer et al., 2009). Slabs with horizontal and rotational support, such as from continuity over the primary supporting beams, can be expected to fail by full depth cracking and rupture of the reinforcement in the hogging regions at the edges of the slab (Nguyen et al., 2015; Jiang and Li, 2018; Usmani and Cameron, 2004).

The current tensile membrane action design methods are based on observations from the Cardington fire tests which did have horizontal restraint at the edges but utilised plain round reinforcing bars. As the bond characteristics of plain round bar and modern deformed bar are different, the rupture of crack-spanning reinforcement can be expected to be influenced by bar type, as bond slip will alleviate the strain localisation at crack locations. It is not known whether failure would have occurred in the Cardington fire tests had deformed reinforcing bars been used. It is possible that the design methods which are based on modelling of the Cardington fire tests are unconservative as they do not correctly represent the rupture of deformed reinforcement (Giroldo and Bailey, 2008).

The strain-rupture behaviour of reinforcing bars at elevated temperature at the locations of flexural cracks, taking account of the bond interaction, has not been

modelled in previous research. One of the challenges in modelling this behaviour is that there was a lack of comprehensive experimental data on the bond stress-slip response of reinforcing bars at elevated temperatures to calibrate such a model.

The rupture of reinforcing bars at the locations of full depth cracks in hogging regions around the boundaries of the slab requires further investigation as this has been demonstrated to be the critical failure mechanism of continuous slabs. The performance of modern deformed bars, relative to plain round bars requires investigation in order to determine the adequacy of design methods which are based on observations from the Cardington fire tests.

2.4 Bond at Ambient Temperature

In order to understand the behaviour of bond at elevated temperature, it is necessary to understand the mechanisms which contribute to the bond between concrete and different types of steel reinforcing bars. It is appropriate to consider how these mechanisms act at ambient temperature before considering the effect that temperature has on them. It is also important to explain the terminology which will be used when describing bond behaviour.

The local relative displacement which occurs between a reinforcing bar and the surrounding concrete is referred to as bond slip. The term bond stress is taken here to refer to the shear stress acting parallel to the axis of the bar at the steel-concrete interface. Normal stress refers to the stress acting radially and perpendicularly to the axis of the bar. These stresses are indicated in figure 2.13, which shows a typical pull-out experiment to characterise the bond stress. Pull-out tests, and other bond testing methods, are described in section 2.5.

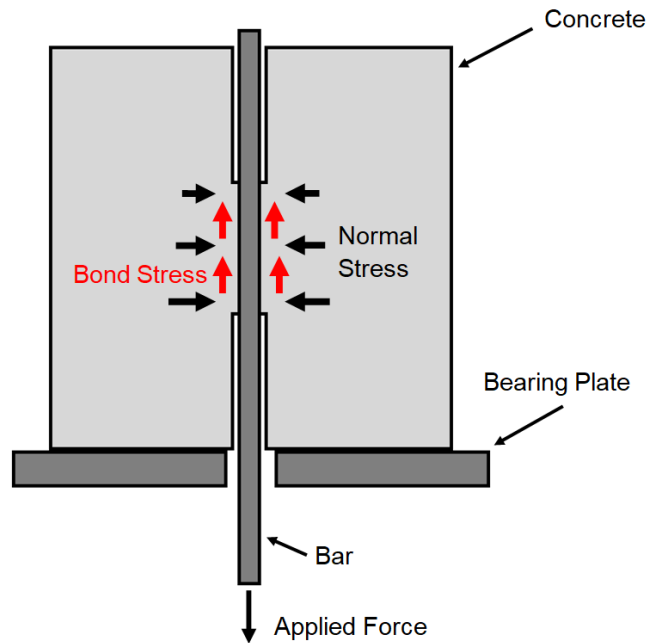


Figure 2.13. Diagram of typical pull-out test showing orientation of bond and normal stresses in the concrete

Bond stress varies along the length of a reinforcing bar. Where reinforcement spans a crack, or at the end of a bonded length, there will be an increase in bond stress near to the edge of the bonded region, as shown indicatively in figure 2.14. The bond stress in figure 2.14 can be seen to reverse as the bar bridges the crack as force is transferred to the concrete on either side in opposite directions. This is relevant to the response of the bar in the research in chapters 5 to 7.

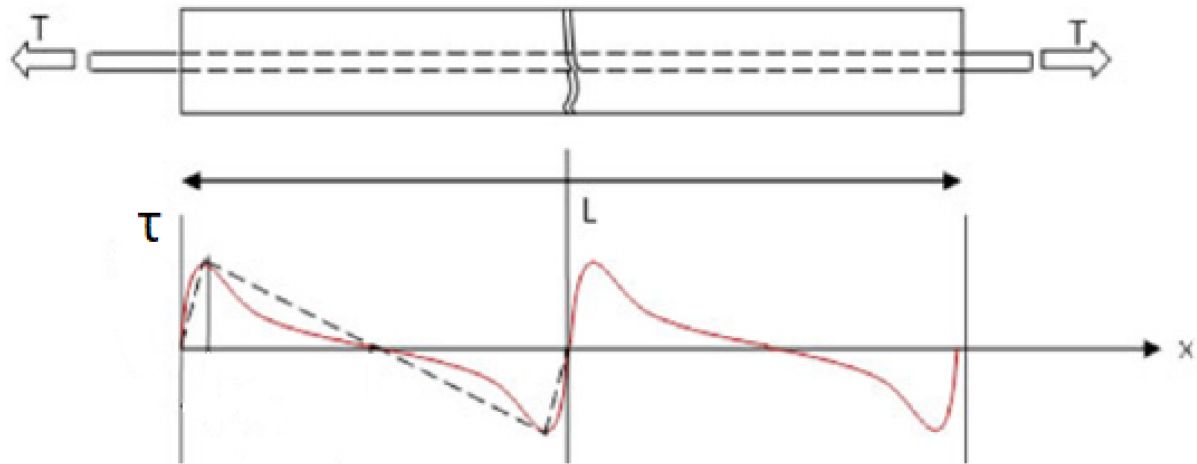


Figure 2.14. Indicative drawing of bond stress distribution in a direct tension test with a central crack in the specimen, figure modified from Solani *et al.* (2013)

The bond between steel reinforcement and concrete is the result of three distinct mechanisms: chemical adhesion, mechanical interlock and friction. The relative importance of these mechanisms depends on the surface geometry of the bar.

The bond strength of a plain bar is mostly provided by chemical adhesion (Mo and Chan, 1996). If the chemical adhesion is overcome and the bar slips then friction forces will develop at the steel-concrete interface. There can be some contribution from mechanical interlock in the bond of a plain bar due to surface roughness but this is not a very significant mechanism for plain bars (Mo and Chan, 1996). Figure 2.15(a) illustrates the bond slip of a plain round bar with a local transition to friction as chemical adhesion is overcome in close proximity to the end of an embedded length.

The bond of deformed bars is primarily due to mechanical interlock between the ribs of the bar and the adjacent concrete. Concrete is crushed below the ribs of the bar as the bar slips. Chemical adhesion is also present for deformed bars, but is

relatively insignificant in comparison to the bond strength contribution of mechanical interlock (Lutz, 1967). After slip begins friction forces will develop and will remain present after the contribution of mechanical interlock has been overcome. Figure 2.15(b) represents the slip of a deformed bar embedded in concrete, showing the crushing of concrete under the ribs of the bar and the local transition from mechanical interlock to friction.

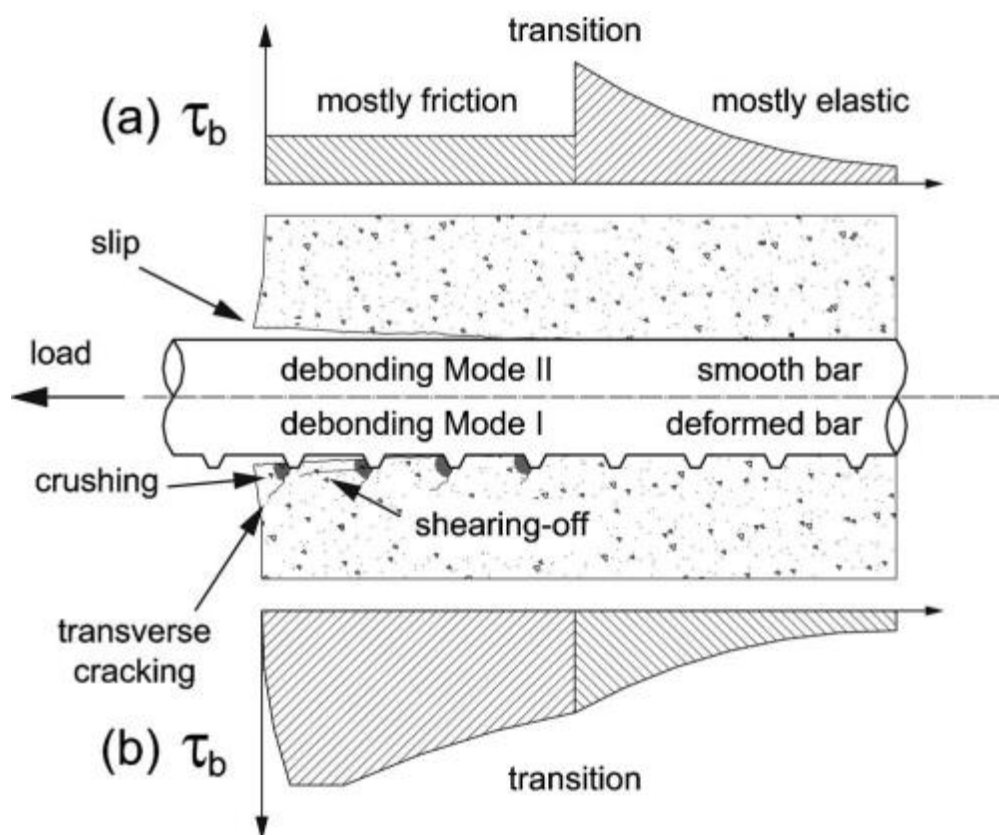


Figure 2.15. Bond mechanisms in (a) plain round bars; (b) deformed bars (Bamonte and Gambarova, 2007)

The ribs of a deformed bar result in not just bond stress but also normal stress due to wedging action of the bar against the surrounding concrete. The normal stress is

indicated in figure 2.13. Han et al. (2019) found that regardless of rib geometry, the most probable bearing angle of wedging action for commonly used concrete is 25° .

The state of the bond between a bar and concrete evolves as slip increases and all of the underlying mechanisms change progressively as a bar slips relative to the surrounding concrete. Fib Bulletin No. 10 (International Federation for Structural Concrete, 2000) characterises bond behaviour in four stages of evolution with increasing slip. These stages are as follows:

Stage 1: at low values of bond stress the chemical adhesion contributes most of bond strength. Concrete is not cracked at this stage but highly localised stresses will arise around the tips of ribs in deformed bars. There will be some micromechanical interaction related to the microscopically rough surface of the bar.

Stage 2: as bond stress increases the chemical adhesion breaks down and slip begins to increase. Large bearing stresses are induced at the ribs of deformed bars and transverse microcracks begin from the tips of the ribs. Wedging action from the ribs remains limited at this stage and there is no concrete splitting.

Stage 3: with further increase in bond stress, longitudinal cracks spread out radially due to wedging action of bar ribs and enhanced by lodged crushed concrete in front of bar ribs. Inclined compressive forces are exerted by the bar ribs and spread into the surrounding concrete. A ring of tension forces forms around the bar to balance the compressive forces as represented by figure 2.16.

Stage 4A (for plain bars): immediately following the breakage of the chemical adhesion, bond strength reduces and remaining force transfer is the result of friction.

At this stage, bond is particularly influenced by concrete shrinkage, bar roughness and transverse pressure.

Stage 4B (for deformed bars): unless there is sufficient confinement to prevent it, if bond stresses are large enough longitudinal cracks will reach the outer surface and splitting will occur. This will result in an abrupt failure of the bond and the bond strength will have been limited by the lack of transverse reinforcement or cover. However, where there is sufficient cover or transverse reinforcement to confine the forces within the concrete, the bond will fail by pull-out of the bar. In this type of failure the concrete around the bar ribs will be sheared off from the rest of the concrete. In this scenario the bond strength is maximised by the presence of confining transverse reinforcement or cover.

The fib Model Code for Concrete Structures (International Federation for Structural Concrete, 2013) presents a local bond model. The model can be used to calculate the relationship between bond stress and slip. This model is described in section 4.1 and used in the proposed modified model which is presented in chapter 4.

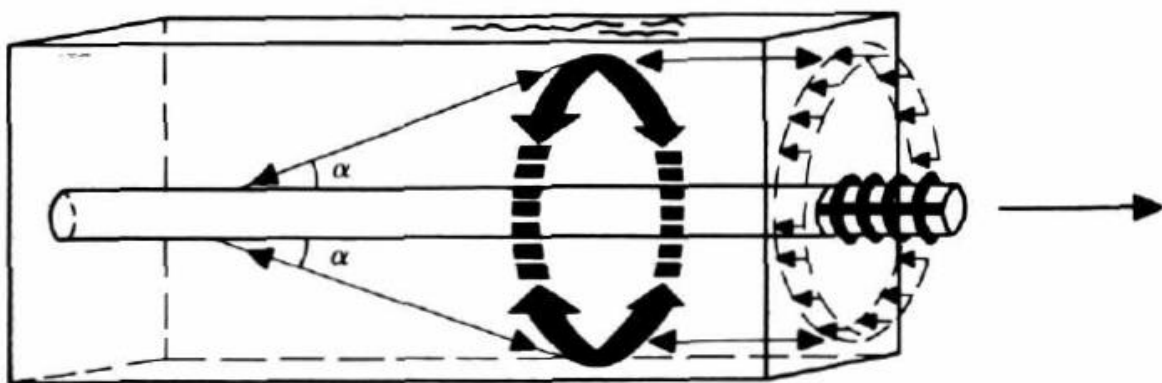


Figure 2.16 Radial components of bond forces balancing against tensile stress rings in the concrete (Tepfers, 1979)

2.4.1 Relevant parameters of bond at ambient temperature

Many factors relating to materials, geometry, casting methods, corrosion and transverse reinforcement are influential on bond performance. This section provides an overview of some of the factors which are of greater relevance to the current research.

2.4.1.1 Bar surface geometry

Bond strength of deformed bars is significantly greater than for plain bars. This has been demonstrated in studies by Mo and Chan (1996) and Kankam (2003). The superior bond of deformed bars is due to mechanical interlock, as discussed in the previous section.

The specific surface geometry of a deformed bar is a leading factor determining bond strength. Surface geometry can be considered mainly in terms of rib height and spacing, although other factors such as the inclination angle of rib surfaces are relevant and are considered in technical standards. The parameters of rib height and rib spacing are combined within a coefficient known as relative rib area, f_R (Rehm, 1961). This coefficient is determined as follows:

$$f_R = \frac{A_R}{\pi \cdot d_b \cdot s_R} \quad (2.5)$$

where: A_R is the axial projected area of a single rib, d_b is bar diameter and s_R is rib spacing. Rehm (1961) found that bond strength is directly proportional to f_R .

2.4.1.2 Bar diameter

Several studies have shown that maximum average bond stress decreases with increasing bar diameter. This trend was identified in the results of pull-out tests by Mathey and Watstein (1961), Walker *et al.* (1997) and Kankam (2003) for various bar diameters. Figure 2.17 compares the average bond stress against slip for 16, 20 and 25mm diameter bars from the results of a study by Kankam (2003). The results shown in figure 2.17 are for the bond of cold-worked reinforcing bars in concrete of 53N/mm² compressive strength.

Walker *et al* (1997) found that for concrete with a compressive strength of 45N/mm² the average bond stress reduced significantly with increased bar diameter. However, with a weaker concrete of 25N/mm² compressive strength, the average bond stress was roughly constant with increasing bar diameter. It appears, therefore, that with relatively low strength concrete the bar diameter may not be a significant factor affecting average bond stress.

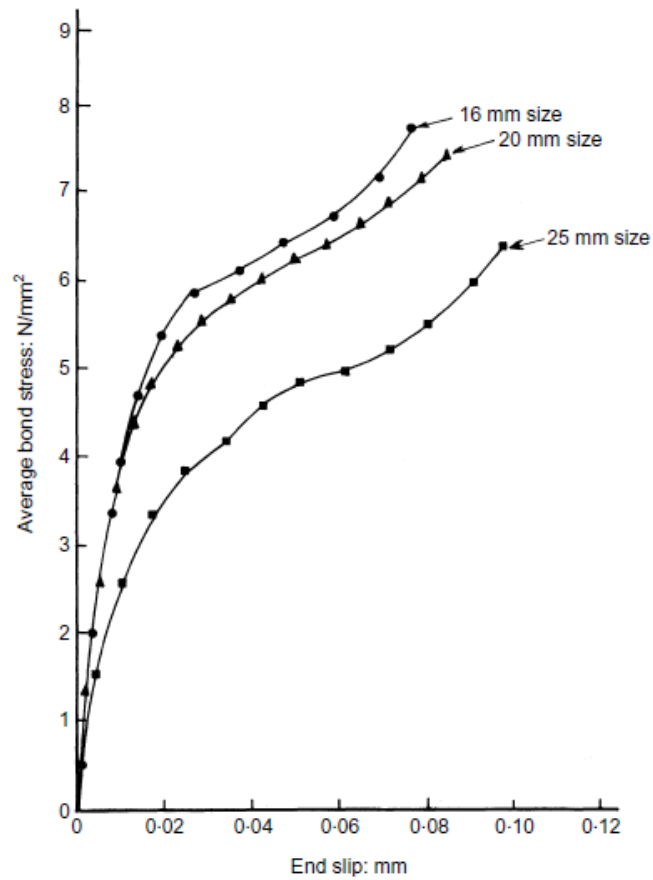


Figure 2.17 Effect of bar diameter on average bond strength, plotted against slip
(Kankam 2003)

2.4.1.3 Concrete strength

Concrete strength is a leading factor in determining bond performance. Higher strength concretes provide better chemical adhesion and also better friction. Hence, higher concrete strength results in increased bond strength for plain round bars.

Mo and Chan (1996) conducted pull-out tests on plain bars embedded in different concrete grades to examine the effect of concrete strength on bond properties. The results of the study by Mo and Chan are shown in Figure 2.18 in terms of stress and slip. These results show a stiffer bond response of plain bars with increased concrete compressive strength.

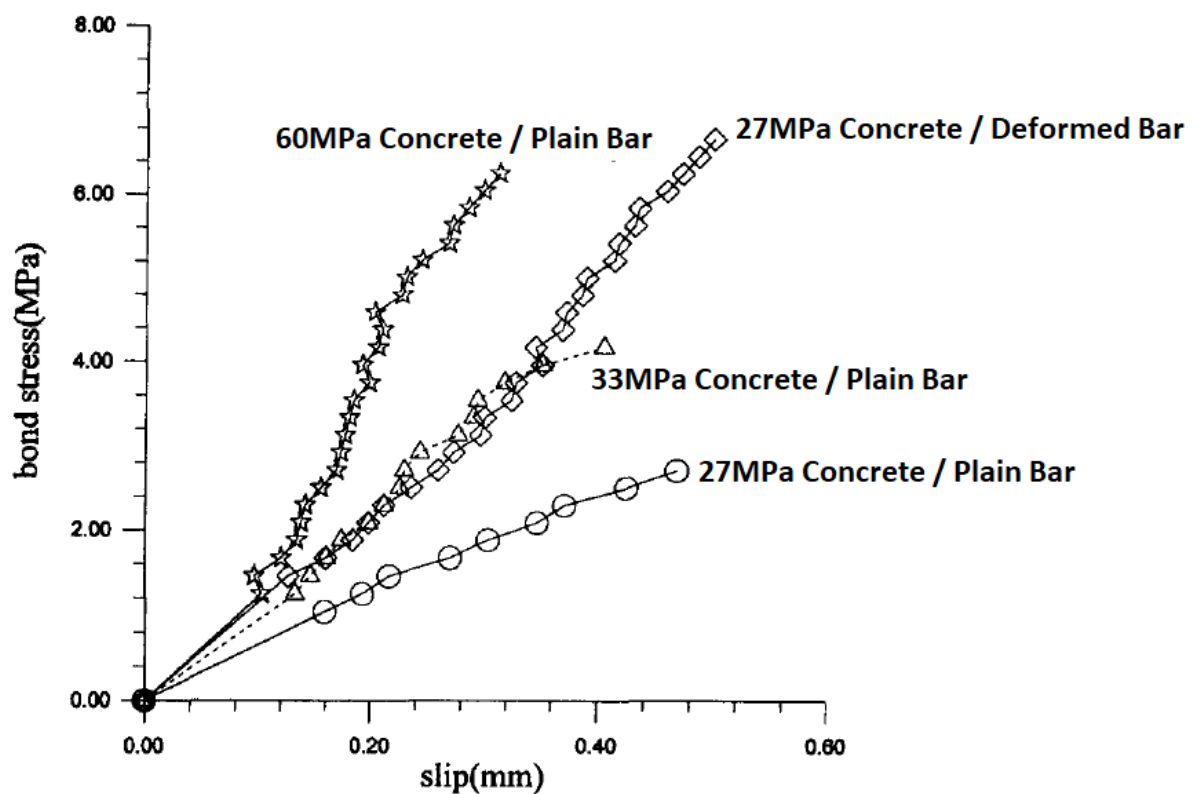


Figure 2.18. Effect of concrete strength on bond stress-slip relationship, modified from Mo and Chan (1996)

Concrete strength is also highly influential in the bond strength of deformed bars. The mechanism of mechanical interlock depends on the compressive and tensile strength of concrete. Rehm (1961) and Reynolds (1982) suggested that bond strength of deformed bars can be considered to be proportional to the square root of the concrete compressive strength, and therefore proportional to concrete tensile strength.

Kankam (2003) conducted pull-out tests with different concrete mixes to investigate the effect of concrete strength on bond. The results of these pull-out tests are shown in Figure 2.19. It is evident that the higher concrete strength resulted in a stiffer bond response at the low slip values considered.

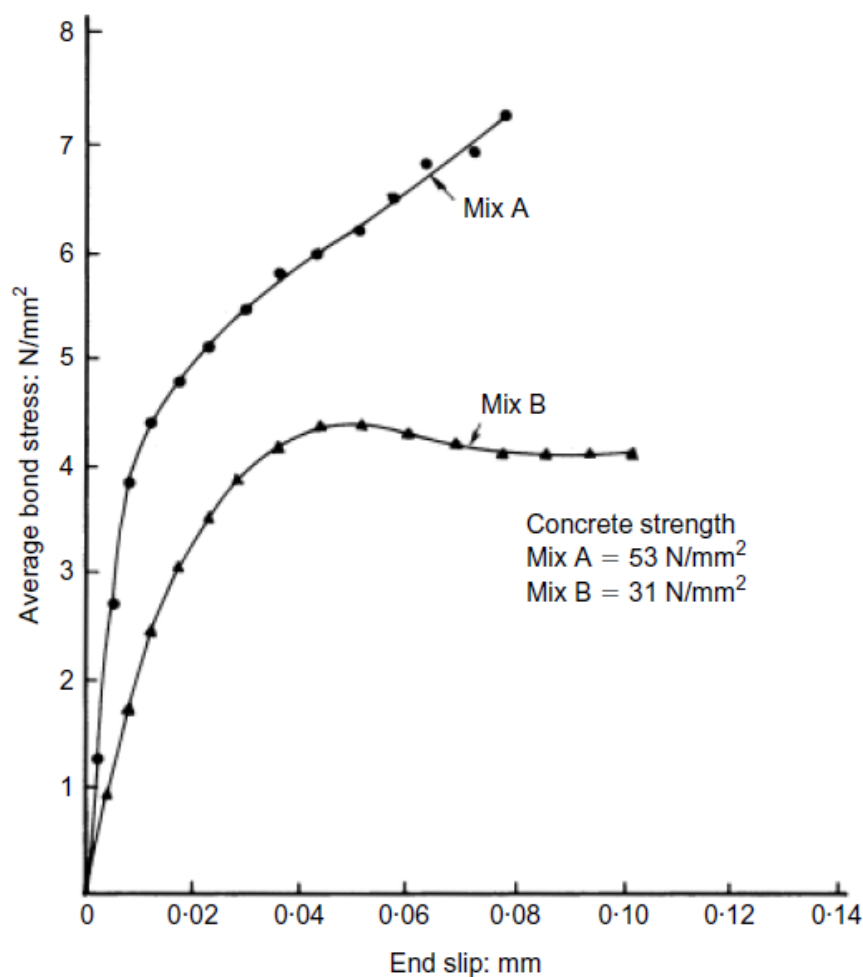


Figure 2.19. Bond stress-slip with different concrete strengths (Kankam, 2003)

2.4.1.4 Concrete cover

The amount of concrete cover over an embedded reinforcing bar determines the type of bond failure which will occur. If concrete cover is insufficient then failure will be by splitting of the concrete. The bond of plain bars will only fail by splitting if the ratio of cover to bar diameter is very small, less than 1.0, which would be unusual. The bond of deformed bars is more prone to splitting failures than with plain bars and splitting can still occur with moderately higher ratios of cover to bar diameter. If concrete cover is sufficient then the failure mode will be by pull-out (Mo and Chan, 1996; Girollo, 2010).

Figure 2.20 is a cross section view showing an idealised force equilibrium condition with a deformed bar embedded in concrete and subjected to pull-out forces. The bearing pressure from the ribs induces an outward radial expansion pressure, resulting in tangential tensile stresses within the surrounding concrete. When the resultant tangential tensile stresses exceed the concrete tensile strength, splitting cracks develop in the concrete cylinder around the bar. If the bearing pressure from the ribs is large enough then the cracking in the concrete cylinder can reach the outer surface and splitting will occur, resulting in bond failure. This splitting could be prevented if concrete cover thickness, C , is sufficient that the splitting cracks do not reach the outer surface, or there is sufficient transverse reinforcement (Tepfers, 1979; Kaa *et al*, 2018).

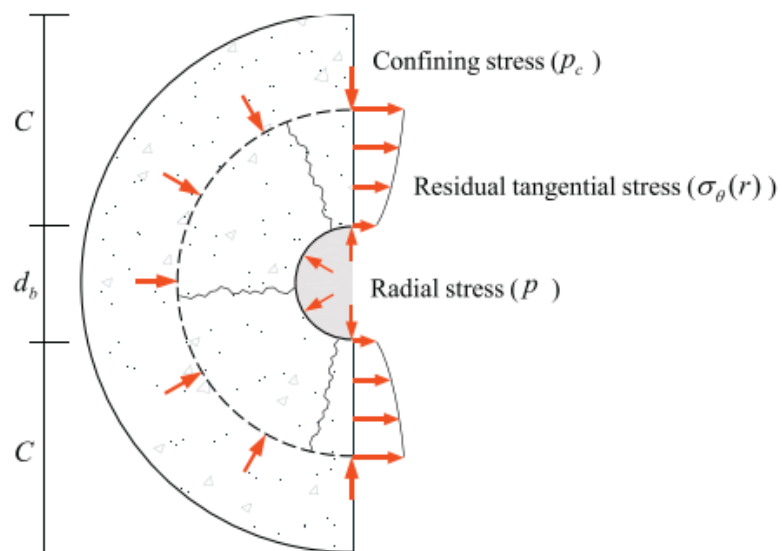


Figure 2.20. Force equilibrium in cracked concrete cylinder, from Ka *et al.* (2018)

2.4.1.5 Bar orientation

Another factor which is known to influence bond performance is the orientation and position of a bar relative to direction in which the concrete is cast. The orientation and position of a bar are relevant because of the processes of sedimentation and settlement. Bars which are orientated perpendicularly to the direction of casting are known to have worse bond than those which are parallel to casting direction. This is due to the formation of voids where micro bubbles settle under the bar (Jirsa *et al.*, 1982).

Menzel (1939) investigated the factors which influence the results of pull-out bond tests by conducting a series of pull-out tests on cylindrical specimens. Bond performance was found to be superior when bars were vertical and cast in the opposite direction to the direction of pull. It was suggested that when cast in this way the concrete would be in intimate contact on the upper face of the ribs but not below the ribs due to settlement. Hence, the bar pulled in the direction of casting would need to slip slightly before the ribs would be brought fully into bearing with the concrete (Menzel, 1939). This effect is not being directly investigated in the current research, but it is relevant to the interpretation of pull-out experiments.

2.5 Bond testing methods

Several different methods have been used to test bond behaviour and each method has its own advantages and disadvantages. The most suitable test method in each case depends on which aspect of bond behaviour is being investigated. Many studies have used their own unique approach, but they can still be categorised by their overall form. This section outlines some of the more commonly used forms of bond test method, and the advantages and disadvantages of each. This is relevant in explaining why the pull-out method was preferred for this research.

The bond test methods which have most commonly been used in the past may be considered as either the transfer type or the anchorage type. Anchorage tests are those in which one end of the bar is stressed in tension and the other end is unstressed, with the concrete providing the equilibrating reaction. In transfer type tests, the bar is gripped at both ends and pulled, putting the bar in tension. No force is directly applied to the concrete in a transfer type test and there is no external reaction force on the concrete. In transfer type tests, the concrete is subject to bond stress as the concrete resists the elongation of the bar (Cairns and Plizzari, 2003).

As was shown in figure 2.14, bond stress distribution varies along a bonded length. In tests of shorter bonded lengths, typically five times bar diameter, the bond stress and slip may be assumed to be uniform, although it will not be exactly uniform (Cairns, 2020). Short bond lengths allow measurement of the local stress-slip behaviour (Cairns and Plizzari, 2003). The local stress-slip behaviour can be extrapolated to longer lengths using numerical modelling.

2.5.1 Pull-out test

Typical pull-out tests are of the anchorage type as they are a clear expression of the idea of anchoring a reinforcing bar. Pull-out tests feature a concrete block, often a cube or cylinder, with a reinforcing bar embedded over a short length. The end of the reinforcing bar is pulled and the concrete usually bears against a surface. Figure 2.21 shows the RILEM standard pull-out test arrangement which is a typical arrangement (RILEM, 1994). There is, however, no universally harmonised and recognised pull-out test arrangement.

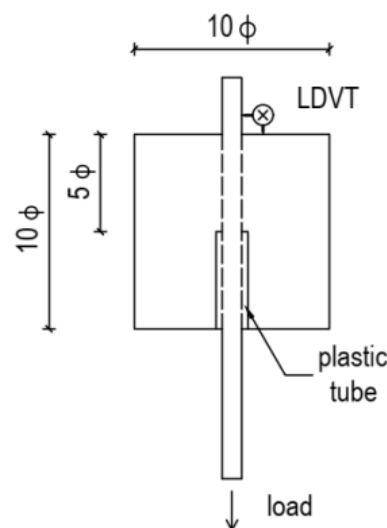


Figure 2.21. The RILEM standard pull-out test (RILEM, 1994; Kucharska and Jaskowska-Lemanska, 2019)

One disadvantage of pull-out tests is that they place the concrete in compression between the bar ribs and the bearing surface. This is not a situation that would be encountered in practical circumstances and will result in unrealistic stress distributions. Pull-out tests are generally provided with enough cover to prevent splitting failures and are more suitable for measuring resistance to pull-out failure rather than splitting failure. One advantage of pull-out tests is that the specimens are

economical to manufacture (Cairns, 2003). It is also relatively straightforward to accurately measure load and slip with these tests. Pull-out tests have had widespread adoption for bond testing.

2.5.2 Double pull-out tests

Another form of pull-out test is the double pull-out test. In these tests, both ends of the reinforcing bar are under tension and the overall elongation of the bar is measured relative to the surrounding concrete. These tests are of the transfer type rather than anchorage type. One disadvantage of transfer type tests is that the force transferred is not directly measured, making it more difficult to determine bond stresses. Although attempts have been made to infer the bar stress by measuring strains, the strain gauges either interfere with the surface of the bar or affect the relationship between cross sections area and surface area. Additionally, the tests are sensitive to the modulus of elasticity of the concrete, and therefore the measurements will be more difficult to replicate (Cairns, 2003).

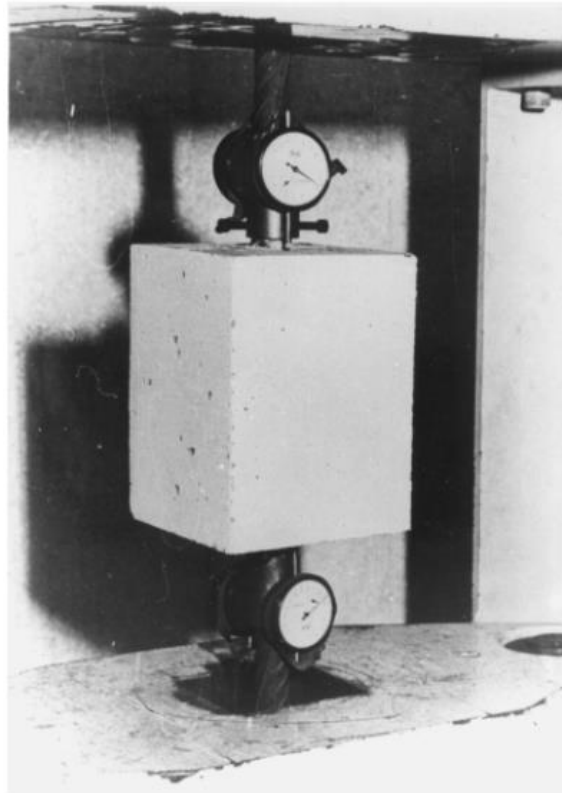


Figure 2.22. A double pull-out test by Kankam (2003)

2.5.3 Beam Test

Another approach to evaluate bond properties is a beam test. This method is more suitable for reinforcement which is subject to bending. The specimen is formed of two half beams which are linked by a steel hinge at the top and by a reinforcement bar in the lower part of the specimen. The specimen is loaded at the top and the specimen is subjected to bending. This method results in a more realistic distribution of stresses in the concrete as the concrete and steel are both in tension. As with pull-out tests, a relatively thick concrete cover is required to ensure a pull-out bond failure rather than splitting (Kucharska and Jaskowska-Lemanska, 2019). Figure 2.23 shows the RILEM standard beam test for evaluation of bond performance. These specimens require confining reinforcement to prevent shear failure (Cairns, 2003).

Snowdon (1970) and Cairns (2003) suggest that this form of test is unnecessarily complex.

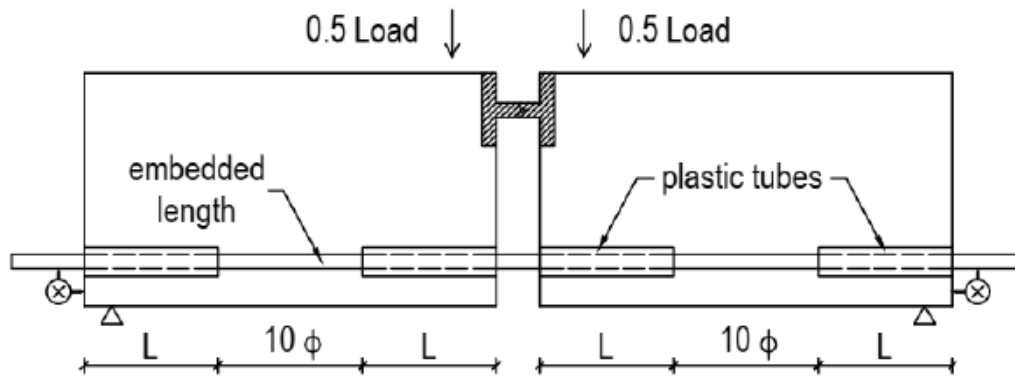


Figure 2.23. The RILEM beam test (RILEM, 1994; Kucharska and Jaskowska-Lemanska, 2019)

2.5.4 Summary of Bond Test Methods

Despite the greater realism in stress distribution of beam tests, traditional pull-out tests are often the preferred choice in bond studies. In pull-out tests it is possible to measure the effect of different conditions such as temperature or bar diameter on the bond stress-slip response during pull-out with a relatively simple test specimen. For this reason, a traditional pull-out test approach is a reasonable choice when conducting a large number of tests to investigate the effect of a range of different parameters.

2.6 The effect of Elevated Temperature on Materials

Elevated temperatures change the mechanical properties of both steel and concrete, as will be explained in the following sections. These effects are relevant for reinforcement bond. The other relevant effects of elevated temperatures are the generation of thermal stresses and thermal expansion of steel and concrete.

This section describes the specific effects of elevated temperature on concrete and steel which are directly relevant to this research. Concrete undergoes complex physicochemical transformations during heating. These transformations occur at different temperatures and substantially change the mechanical properties of the concrete. The behaviour of any concrete at elevated temperatures is dependent on the mix constituents and proportions.

Concrete has both advantages and disadvantages as a structural material when subject to fire. Two advantages of concrete in a fire are that it is non-combustible and that it has good insulating properties. Some disadvantages of concrete when subject to fire are that its mechanical properties will deteriorate as temperature increases and that it can suffer explosive spalling, resulting in material loss (Buchanan, 2017; Khoury, 2000).

The compressive stiffness of concrete is relatively unchanged at temperatures up to 450°C, but reduces significantly by 600°C due to physicochemical transformations. This is based upon measurements of concrete tested while hot (Weigler and Fischer, 1972). The effect of temperature on concrete compressive strength and tensile strength are explained in sections 2.61 and 2.62 respectively.

Steel tends to perform poorly in fire unless it is protected. Some of the main reasons for this are the high thermal conductivity and the thinness of structural elements relative to other structural materials (Buchanan, 2017). In the context of composite structures, steel reinforcement generally benefits from insulation of the surrounding concrete. This section provides an overview of the more relevant effects of elevated temperature on steel and concrete.

2.6.1 Concrete Compressive Strength

Concrete compressive strength decreases with increasing temperature, but it does not follow the simple, monotonic, declining trend given by design codes such as Eurocode 4 Part 1-2 (BSI, 2005b). The relationship between concrete compressive strength and temperature depends greatly on the specific mix constituents and proportions (Khoury, 2000). The retention in concrete compressive strength with increasing temperatures for two particular concretes tested by Castillo and Durrani (1990) are shown in figure 2.24. These concretes have compressive strengths of 33.1MPa and 62.8MPa. High strength concretes are more severely affected by the build-up in pore pressure during heating due to their low permeability relative to lower strength concretes (Kodur, 2005).

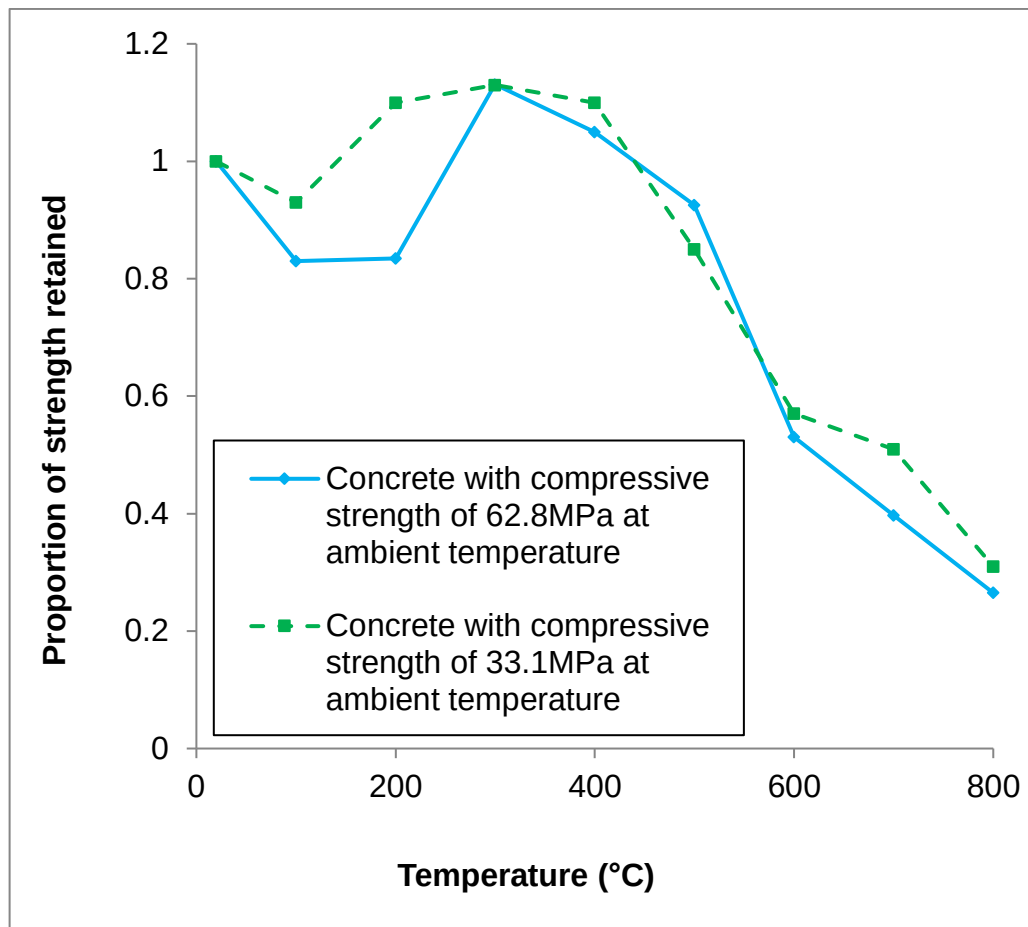


Figure 2.24. Change in concrete compressive strength with temperature from experiments by Castillo and Durrani (1990) for two particular concretes with ambient compressive strengths of 62.8MPa and 33.1MPa

Compressive strength reaches an initial minimum at around 80°C for unsealed concrete. This is attributed to van der Waals' forces weakening as water molecules expand and CSH layers are pushed further apart. This initial strength loss is mostly reversible upon cooling (Castillo and Durrani, 1990; Khoury, 2000).

As temperatures increase further, concrete can regain strength and reach a maximum compressive strength in the region of 200 to 300°C. The compressive strength in this temperature range can exceed the ambient strength. This strength increase is attributed to cement gel stiffening or an increase in surface forces

between gel particles due to adsorbed moisture removal (Castillo and Durrani, 1990; Khoury, 2000).

Concrete compressive strength then decreases with further increases in temperature and strength rapidly declines in the range of 400°C to 600°C. In this temperature range, the cement paste dehydrates and gradually disintegrates (Castillo and Durrani, 1990). The shrinkage of cement paste and expansion of aggregate results in a weakening of the bond between aggregate and cement paste, reducing strength. At 800°C, compressive strength drops to around 30% of its ambient value. This is a generalised description of concrete compressive strength with increasing temperatures and, as previously mentioned, the actual trend for a specific concrete is dependent on the mix.

2.6.2 Concrete Tensile Strength

The tensile strength of concrete also decreases with increasing temperatures. The decrease in tensile strength is, however, monotonic and near linear with increasing temperature. Eurocode 2 Part 1-2 (BSI, 2019) includes a reduction factor for the characteristic tensile strength of concrete, $k_{ct}(\theta)$, at elevated temperatures, which is shown in figure 2.25.

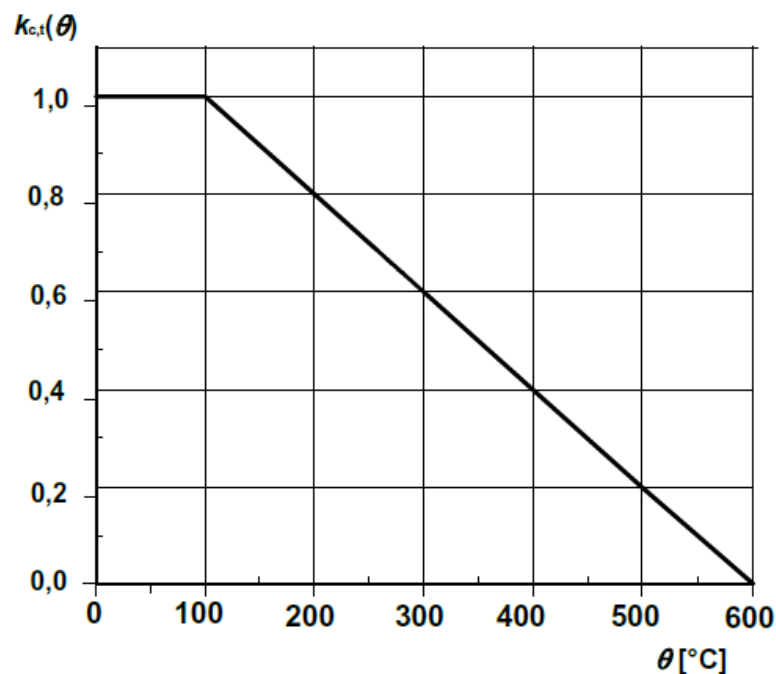


Figure 2.25. Reduction factor for the characteristic tensile strength of concrete, $k_{ct}(\theta)$, at elevated temperatures from Eurocode 2 Part 1-2 (BSI, 2019).

Figure 2.26 shows the relationship between concrete tensile strength and increasing temperatures for one particular 61.1MPa concrete, as tested under direct tension by Noumowe (1996). At temperatures up to around 400°C, the retention of tensile and compressive strength of concrete are dependent on aggregate type. Above these temperatures the reduction in strength follows quasi-identical behaviour with different aggregates (Hager *et al.*, 2015).

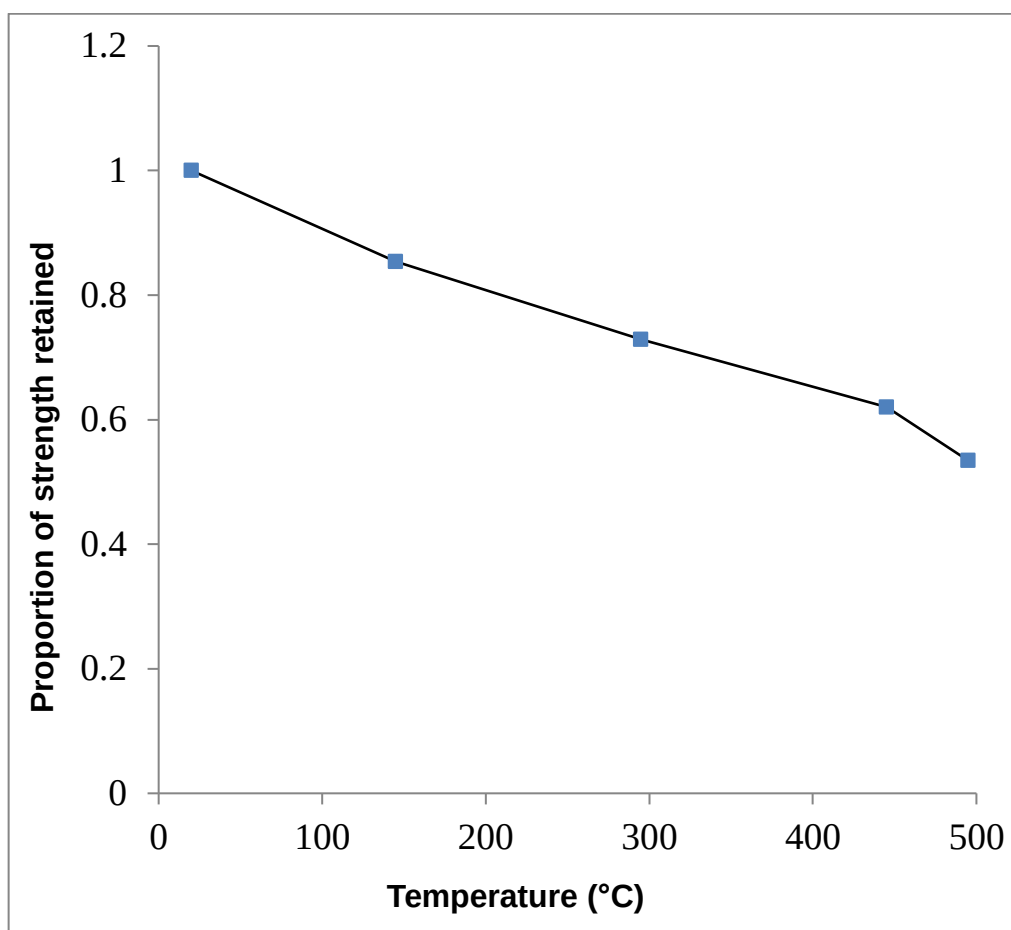


Figure 2.26. Change in concrete tensile strength with temperature from experiments
by Noumowe (1996)

2.6.3 Steel

Steel loses strength with increasing temperatures. The yield strength and ultimate strength factors according to BS EN 1994 Part 1-2 (BSI, 2005b) for hot rolled reinforcing bars are shown in figure 2.27. With the exception of cold worked steel, the loss of strength is largely recoverable after cooling from temperatures not more than around 700°C. The yield strength of steel decreases by half at a temperature of around 550°C. The reduction in steel strength with elevated temperature is a factor in the potential rupture of reinforcement in reinforced concrete and composite slabs.

Reinforcing steel should not typically reach temperatures of several hundreds of degrees Celsius in a fire scenario unless spalling or cracking has reduced or removed concrete cover. This would expose the steel to more direct heat transfer from the fire and could result in a substantial weakening of the steel.

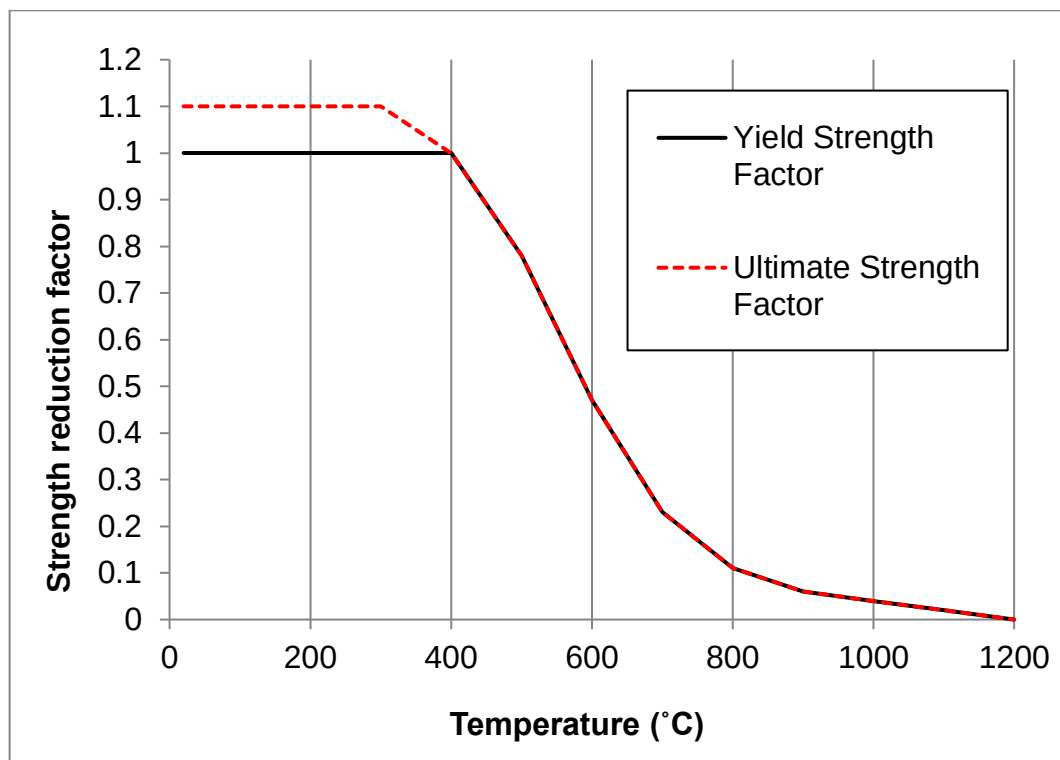


Figure 2.27. Yield strength and ultimate strength factors according to Eurocode 4 Part 1-2 (BSI, 2005b) for hot rolled reinforcing bars.

The relationship between the elastic modulus of steel and temperature up to 600°C according to Eurocode 4 Part 1-2 (BSI, 2005b) is shown in figure 2.28. Figure 2.29 shows the Poisson's ratio of steel with increasing temperature according to experimental research by Barh *et al.*, (2012). Figure 2.29 is limited to 500°C because measurements over this temperature were not reliable.

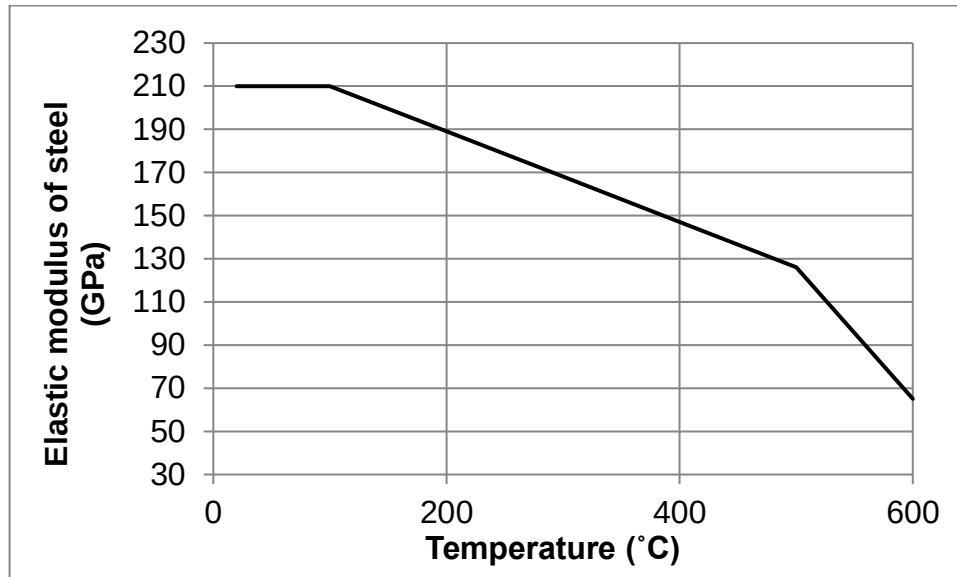


Figure 2.28. Relationship between the elastic modulus of steel and temperature up to 600°C, from Eurocode 4 Part 1-2 (BSI, 2005b)

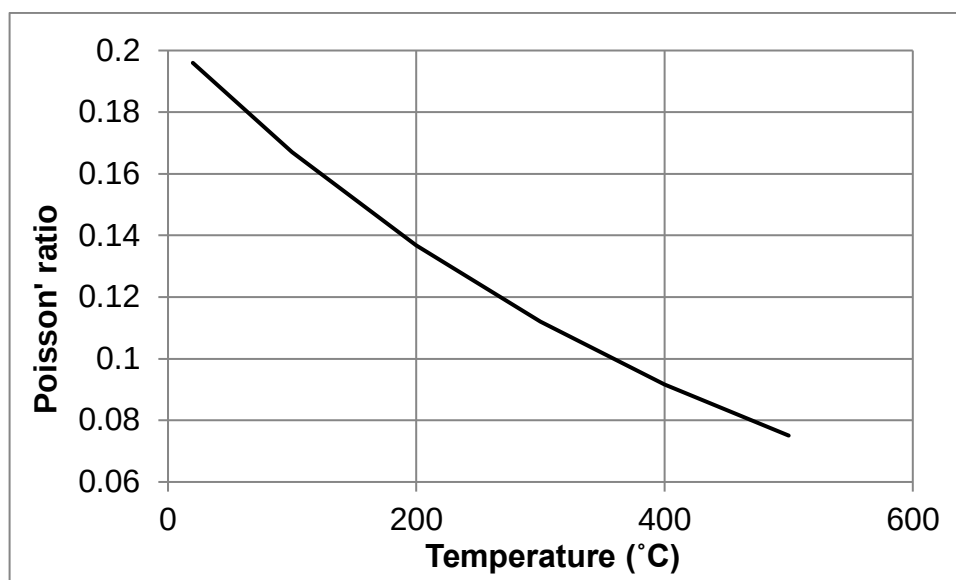


Figure 2.29. Relationship between Poisson's ratio of steel and temperature (Barh *et al.*, 2012)

2.6.4 Strain of Concrete and Steel at Elevated Temperatures

The deformation of a structure in the fire limit state is a fundamental consideration in structural fire analysis. Buchanan and Abu (2017) describe the deformation of materials in terms of a change in strain, $\Delta\varepsilon$, consisting of four strain components:

$$\Delta\varepsilon = \varepsilon - \varepsilon_i = \varepsilon_\sigma(\sigma, T) + \varepsilon_{th}(T) + \varepsilon_{cr}(\sigma, T, t) + \varepsilon_{tr}(\sigma, T) \quad (2.6)$$

where the term ε is total strain at time t , ε_i is initial strain at time $t = 0$, the term $\varepsilon_\sigma(\sigma, T)$ is mechanical strain and is a function of applied stress, σ , and temperature, T , the term $\varepsilon_{th}(T)$ is thermal strain and is a function of temperature, the term $\varepsilon_{cr}(\sigma, T, t)$ is creep strain and is a function of applied stress, temperature and time, and the term $\varepsilon_{tr}(\sigma, T)$ is transient strain and is a function of applied stress and temperature.

Mechanical strain is the strain which is generated by the stresses in the structural element. In the numerical modelling parts of this research, these strains are based on temperature dependent stress strain curves, as described in chapter 5. Creep strain is a deformation which typically occurs over long periods of time. Creep strain is usually not explicitly included in fire engineering calculations due to the complexity and a lack of sufficient data for implementation (Buchanan and Abu, 2017). Creep strain is not expected to be a significant factor in the very localised structural behaviour of interest to this research and it is not considered further.

Thermal strain is also referred to as thermal expansion and is explained further in section 2.6.5. Transient strain is sometimes referred to as Load Induced Thermal Expansion (LITS), and is explained in section 2.6.6.

2.6.5 Thermal Expansion of Steel and Concrete

Thermal expansion, or thermal strain, occurs in materials, including concrete and steel, during heating. This section refers to free thermal expansion, which is a strain which a material experiences under heating without any mechanical load. As concrete is a heterogeneous material, its thermal expansivity is the cumulative effect of individual components. Thermal expansion of concrete is dependent on a range of factors, including mix proportions and aggregate type (Malik et al., 2021).

In addition to differential thermal expansion between aggregate and cement paste within concrete, bond is substantially influenced by the differential thermal expansion between concrete and steel. Figure 2.30 shows the thermal strain of types of concrete and reinforcing steel with increasing temperature relative to their 20°C state, from Eurocode 2 Part 1-2 (BSI, 2019). The thermal expansivity of different concrete mixes will differ from those in figure 2.30 and no single curve will accurately represent all mixes. As is evident from figure 2.30, the thermal expansion of concrete is known to be highly dependent on aggregate type.

Under around 100°C, the thermal strain of steel and concrete are relatively similar. At elevated temperatures under around 450°C, the thermal strain of the reinforcing steel is greater than that of the surrounding concrete. Such conditions could be expected to result in a confining pressure on the bar from the surrounding concrete.

Over around 450°C, the thermal strain of siliceous concrete begins to exceed that of steel. It is also important to note that as temperature increases even further the differential in thermal strain becomes increasingly large. It would be logical to expect

that this would result in a physical separation of the steel and concrete surfaces at the bond interface.

Although thermal strain will affect the global structural behaviour of a two way spanning slab under elevated temperatures, the focus of this research is on the rupture of the crack-spanning reinforcement in regions which are dominated by hogging and flexural crack development. The thermal strain effects which are of particular interest to this research are, therefore, the differential thermal expansion effects between steel and concrete at the bond interface. Thermal strain effects are present in the results of the elevated temperature bond tests in Chapter 3 as they are conducted at elevated temperature. The effects of thermal strain on the bond are, therefore, translated into the numerical modelling work through the definition of the bond interaction, as explained in Chapter 5.

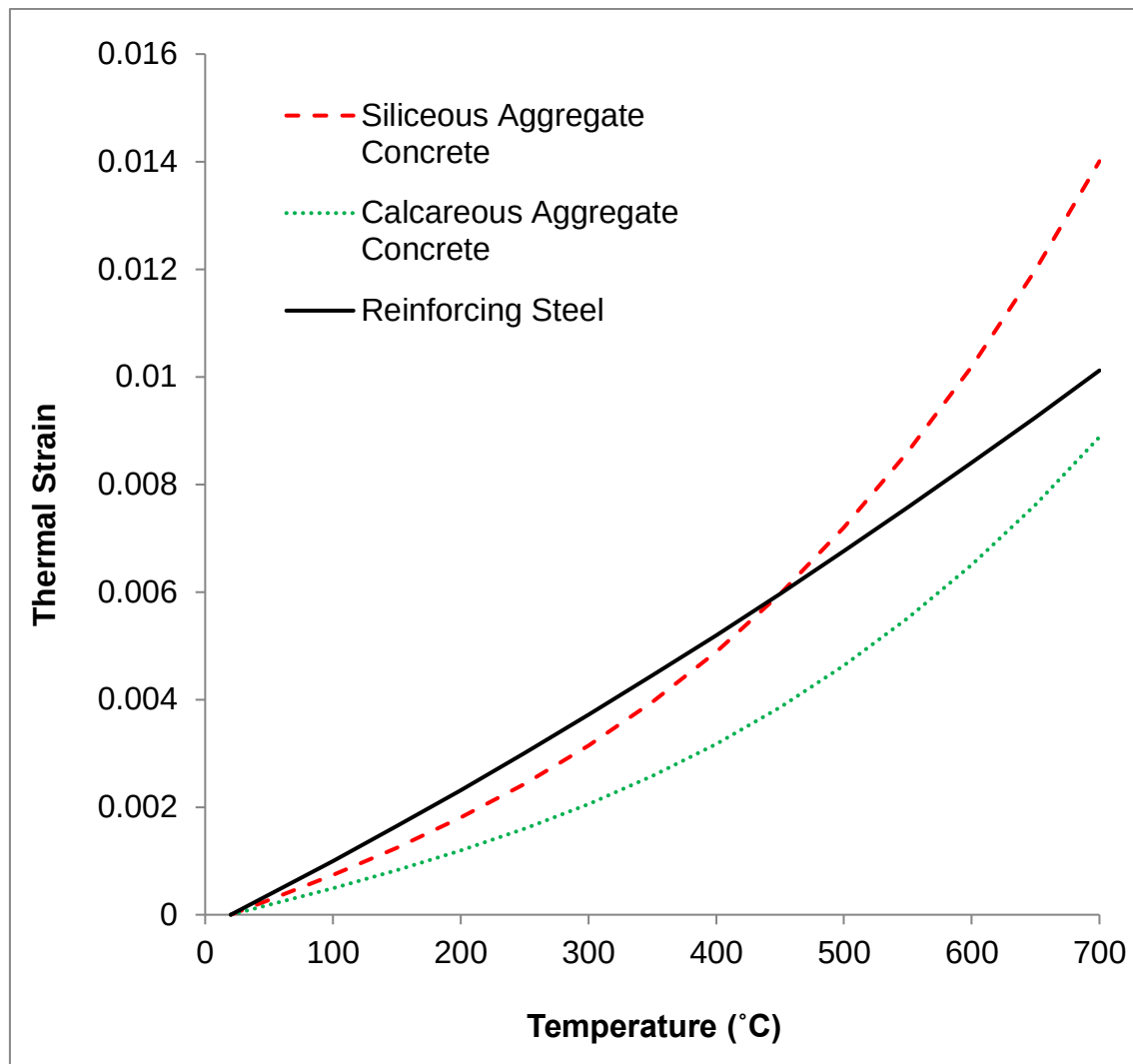


Figure 2.30. Thermal strain of reinforcing steel and types of concrete with increasing temperature relative to their 20°C state, from Eurocode 2 Part 1-2 (BSI, 2019)

2.6.6 Load Induced Thermal Strain

Load induced thermal strain (LITS) applies only to concrete and it is integral to the behaviour of concrete in fire (Law et al., 2009). Law (2010) defined LITS as "the difference between the free thermal expansion of concrete and the thermal expansion when the same concrete is heated under a level of pre-stress". Experimental research has demonstrated that concrete specimens which are mechanically loaded exhibit LITS upon virgin heating (Torelli et al., 2016). The extent of LITS increases with increasing temperature and increasing compressive stress and it is mostly irrecoverable after the first heating of concrete (Torelli et al., 2016).

LITS is a term which refers to the collective effect of several different strain components. These components are transitional thermal creep, basic creep, drying creep and changes in elastic strains due to changes in the elastic modulus as temperature increases (Law et al., 2009; Gernay and Franssen, 2012). LITS is predominantly due to chemical and physical reactions which take place in the cement paste at temperatures up to around 400°C and also thermal incompatibility between aggregate and cement paste at higher temperatures (Torelli et al., 2016).

Eurocode 2 Part 1-2 (BSI, 2019) accounts for LITS implicitly, by artificially increasing the strain parameters in the compressive stress-strain curve for concrete with increasing temperature. This has the effect of decreasing the tangent modulus with increasing temperature. One inherent limitation of implicit modelling of LITS is that the same mechanical strain is calculated for a given stress-temperature state regardless of whether the concrete is heated prior to loading, or loaded prior to heating, and this is known to be incorrect (Gernay and Franssen, 2012). Another limitation of implicit models is that transient creep strain is recovered during

unloading because the elastic modulus for unloading is taken as the initial tangent of the stress-strain curve. Additionally, implicit models are particularly unrepresentative in the cooling phase (Gernay and Franssen, 2012). Several authors have proposed concrete models explicitly integrating a term for LITS, and they do not have the same inherent limitations (Anderberg, 1976; Nielsen et al., 2002; Terro, 1998). These explicit models can be incorporated into numerical modelling if required.

The constitutive model of Eurocode 2 has, however, proved to yield quite satisfactory results in structural calculations, although it has mostly been applied under ISO fire conditions where temperature only increases over time (Gernay, 2012). Lange and Jansson (2014) found that the implicit formulation in the Eurocodes performs well in conditions where compressive loading of concrete during heating is relatively small, but not with increased compressive loading.

Although tensile membrane action is clearly dominated by tensile stresses, rather than compressive stresses, LITS may still be present in regions of the slab. There will be a compressive zone in the hogging regions around the edges of a horizontally restrained slab, and LITS may form in this zone.

The focus of this research is on the rupture of the crack-spanning reinforcement around the edges of the slab. The modelled specimen in Chapter 5 is under-reinforced to ensure that the flexural response is dominated by flexural cracking, rather than either compressive strain or crushing. Additionally, the model is progressively loaded, with no unloading phase, and the cooling phase is outside the scope of this research. It is, therefore, considered reasonable that LITS is represented implicitly, through the Eurocode 2 constitutive model, in the numerical modelling work in chapters 5 to 7 of this research for simplicity.

2.7 Modelling concrete

2.7.1 Concrete Damage Plasticity Model

Concrete behaves in a brittle manner and the two common failure modes of concrete are compressive crushing and tensile cracking. Constitutive models, such as the 'concrete damage plasticity' model, have been developed to provide a capability of analysing the complex response of quasi-brittle materials including concrete. The concrete damage plasticity model which is provided in Abaqus finite element software is the focus of this section as it is utilised in the research described within chapter 5. This section describes the features of the 'concrete damage plasticity' model which are relevant to the input parameters which were used in chapter 5.

The concrete damaged plasticity model in Abaqus is a continuum, plasticity-based, damage model for concrete which assumes that the two main failure mechanisms are tensile cracking and compressive crushing (Dassault Systèmes, 2012). The model utilises a yield function developed by Lubliner *et al.* (1989), including modifications by Lee and Fenves (1998) accounting for differences in the evolution of strength in tension and compression. The yield function is defined by Equation 2.7:

$$F = \frac{1}{1-\alpha} (\bar{q} - 3\alpha\bar{p} + \beta(\bar{\epsilon}^{pl})\langle\hat{\sigma}_{max}\rangle - \gamma\langle-\hat{\sigma}_{max}\rangle) - \bar{\sigma}_c(\bar{\epsilon}_c^{pl}) = 0 \quad (2.7)$$

Where the term F is a scalar function of invariants of the stress tensor (Lee and Fenves, 1988). The parameters α , β and γ are defined by equations 2.8, 2.9 and 2.10 respectively.

$$\alpha = \frac{\left(\frac{\sigma_{b0}}{\sigma_{c0}}\right)^{-1}}{2\left(\frac{\sigma_{b0}}{\sigma_{c0}}\right)^{-1}} \quad (2.8)$$

$$\beta = \frac{\bar{\sigma}_c(\bar{\varepsilon}_c^{pl})}{\bar{\sigma}_t(\bar{\varepsilon}_t^{pl})}(1 - \alpha) - (1 + \alpha) \quad (2.9)$$

$$\gamma = \frac{3(1-K_c)}{2K_c-1} \quad (2.10)$$

The ratio of initial equibiaxial to initial uniaxial compressive yield stress (σ_{b0}/σ_{c0}) has a default value in Abaqus of 1.16. Monte, Kalaba and Bamonte (2017) found that at temperatures under 600°C the value of σ_{b0}/σ_{c0} is in the range of 1.1 to 1.3. The hardening variables $\bar{\varepsilon}_t^{pl}$ and $\bar{\varepsilon}_c^{pl}$ are the tensile and compressive equivalent plastic strains. $\bar{\sigma}_t(\bar{\varepsilon}_t^{pl})$ and $\bar{\sigma}_c(\bar{\varepsilon}_c^{pl})$ are the effective cohesion stresses in tension and compression, respectively. The parameters $\bar{p}$ and $\bar{q}$ are the hydrostatic pressure stress and the Mises equivalent effective stress, respectively.

The parameter K_c is the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian and it defines the shape of the yield surface in the deviatoric plane. Monte, Kalaba and Bamonte (2017) found little temperature dependency in K_c , with values within the range of 0.6 to 0.67 for temperatures from 20°C to 600°C.

The concrete damage plasticity model includes a Drucker-Prager hyperbolic function of non-associated potential plastic flow. This flow potential function, G , is defined according to equation 2.11:

$$G = \sqrt{(\epsilon\sigma_{t0}\tan\psi)^2 + \bar{q}^2} - \bar{p}\tan\psi \quad (2.11)$$

The parameter σ_{t0} is the uniaxial tensile stress. The parameter (ϵ) is the eccentricity of plastic flow potential, which defines the rate at which the function of hyperbolic flow potential reaches its asymptote. The dilation angle (ψ) parameter defines the inelastic change in concrete volume within the Drucker-Prager hyperbolic function.

Genikomsou and Polak (2015) studied the effect of dilation angle on the deflection response of concrete slabs at ambient temperature. They determined a range of 20° to 42° and used 40°. Wahid *et al.* (2019) found that 35° produced the most accurate response of the values considered.

Use of the viscosity parameter, μ , enables viscoplastic regularisation of the concrete constitutive equations. A value which is small, relative to the characteristic time increment, can improve the rate of convergence without compromising results. Wahid *et al.* (2019) recommended that a value of 0.0001t could be used in high temperature analysis, with 0.001t also suitable if results are in good agreement with test results.

2.7.2 Modelling of Concrete at High Temperatures

When modelling concrete at elevated temperatures, it is important to incorporate the temperature dependency of the thermal and mechanical material properties of concrete. The definition of the concrete stress and strain relationship is known as a constitutive model. Thermal strain does not need to be included within the constitutive model because it is not dependent on the stress-state of the material. The constitutive model used in the numerical modelling in this research is the model presented in Eurocode 2 Part 1-2 (BSI, 2019). The ascending region of the relationship between stress, σ , and strain, ε , is defined by the following equation:

$$\sigma = \frac{3\varepsilon f_{c,\theta}}{\varepsilon_{c1,\theta} \left(2 + \left(\frac{\varepsilon}{\varepsilon_{c1,\theta}} \right)^3 \right)} \quad (2.12)$$

where $f_{c,\theta}$ is the compressive strength of concrete at temperature θ , and $\varepsilon_{c1,\theta}$ is the strain corresponding to the point of peak compressive stress. As permitted by Eurocode 2, the descending branch is considered as linear in the numerical models

used in Chapters 5 to 7 of this research. The concrete compressive stress-strain relationship used in this research is shown in figure 2.31. Eurocode 2 specifies the modulus of elasticity of concrete as the secant modulus of the compressive stress-strain curve in the range 0 to $0.4f_{cm}$.

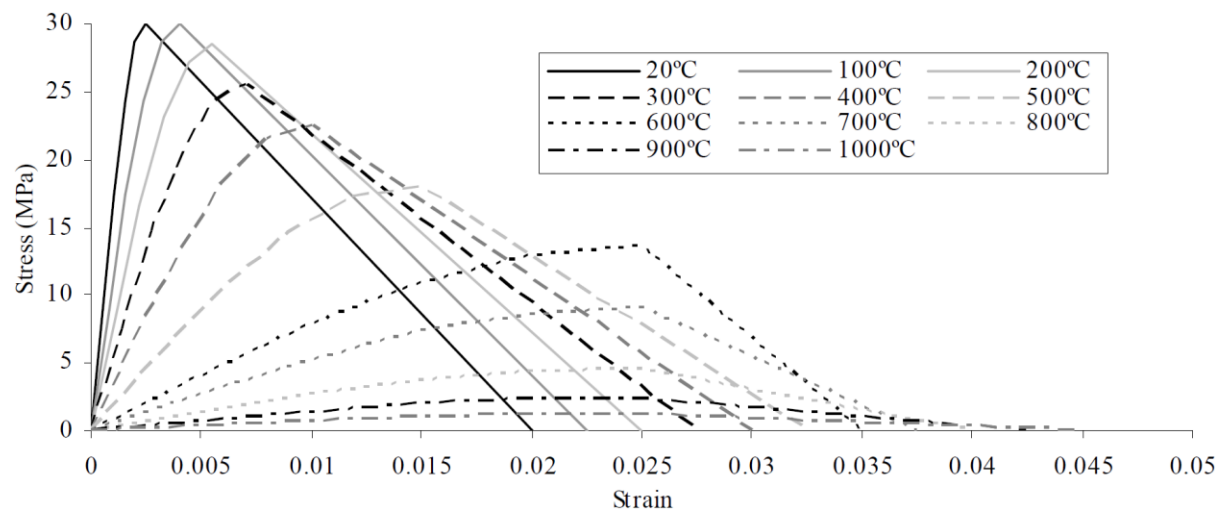


Figure 2.31. Temperature dependent constitutive stress-strain curves for concrete in compression, according to Eurocode 2 Part 1-2 (BSI, 2019), sourced from Law (2010).

Aslani and Bastami (2011) proposed a relationship between concrete tensile strength and temperature, based on regression analysis of several sets of experimental data. The Eurocode 2 model for concrete tensile strength with increasing temperature appears to significantly underestimate concrete tensile strength, presumably for conservatism in design, with no concrete tensile strength above 600°C which contradicts experimental data (Aslani and Bastami, 2011; Noumowe et al., 1996; Xu et al., 2003). As tensile crack development is a key aspect of the numerical modelling research presented in Chapters 5,6 and 7, the accuracy of the tensile strength definition is of particular importance. For this reason, the

relationship between concrete tensile strength and temperature proposed by Aslani and Bastami (2011) was adopted in the numerical models.

From the perspective of thermal analysis, or heat transfer analysis, it is necessary to include the temperature dependency of thermal conductivity, specific heat capacity and density in the model. A description of the temperature dependent properties of the thermal conductivity, specific heat capacity and density of concrete were described in section 2.1.5. The temperature dependent specific heat capacity and density are included in the numerical modelling method in this research as per the series of equations in section 3.3.2 of Eurocode 2 Part 1-2 for concrete with siliceous aggregates. The thermal conductivity of concrete was taken as per the recommendation of the UK National Annex to Eurocode 2 Part 1-2 for normal weight concrete, as given by the following equation:

$$\lambda_c = 1.36 - 0.136 \left(\frac{\theta}{100} \right) + 0.0057 \left(\frac{\theta}{100} \right)^2 \quad \text{for } 20^\circ\text{C} \leq \theta \leq 1200^\circ\text{C} \quad (2.13)$$

The different forms of strain which are present in concrete and steel at elevated temperatures are described in sections 2.6.4 to 2.6.6. The incorporation of temperature dependent mechanical properties for both concrete and steel in the numerical modelling method is described in chapter 5.

2.8 The effect of elevated temperature on bond

Bond strength between concrete and steel reinforcement is known to reduce with increasing temperature. This has been found to be true for both deformed bars and plain bars (Diederichs and Schneider, 1982; Hertz, 1982; Morley and Royles, 1983). As the mechanisms which govern the bond of deformed and plain reinforcement are different, the effects of elevated temperature on the bond of these bar types are also different.

It is not known from previous research how the behaviour of modern reinforcing materials will be affected by elevated temperatures as technical specifications of bar surface geometry and mechanical properties have changed in recent decades (BSI, 1978; 2005c). There is a particular lack of knowledge of how the stiffness of the bond response is affected by elevated temperatures. This effect on bond stiffness is particularly relevant to the research described in chapters 5 to 7.

2.8.1 Previous elevated temperature bond testing

There have been a few previous studies which have investigated bond at elevated temperatures. The more relevant studies are described here. Differences in test methods and materials mean that the comparison must be made in the context of these differences.

2.8.1.1 Morley and Royles

Morley and Royles (Morley, 1982; Morley and Royles, 1983; Royles and Morley, 1983) conducted a series of elevated temperature pull-out tests to investigate the effect of elevated temperatures on the bond between steel reinforcement and concrete. The test specimens were cylindrical with a height of 300mm. The diameter of the specimens was varied between specimens to consider different cover depths.

The steel and concrete were bonded only over a 32mm length in the centre of the specimen.

Each specimen was heated in a custom furnace, as shown in figure 2.32. Tests were conducted on specimens under the four conditions listed:

1. stress applied during heating, then loaded to failure when hot
2. stress applied during heating, then loaded to failure after air cooling for 24 hours
3. no stress applied during heating, then loaded to failure when hot
4. no stress applied during heating, then loaded to failure after air cooling for 24 hours

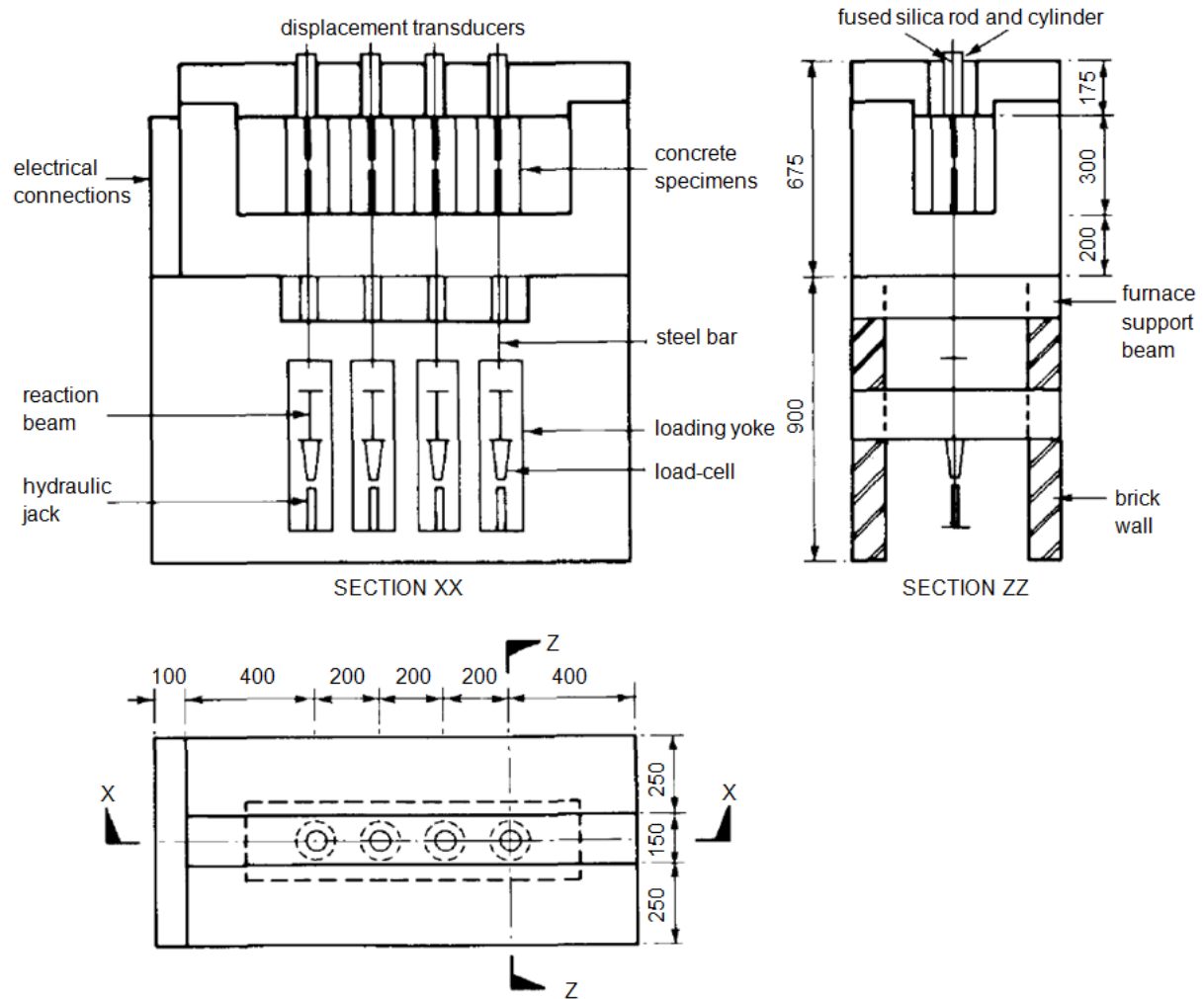


Figure 2.32. Test arrangement by Morley and Royles (1983), drawing modified for clarity

Specimens containing 16mm deformed reinforcing bars were tested in each of the four conditions. Plain bars of 16mm diameter were also tested in condition 2 only. A 55mm cover depth was used for each of these four conditions. Additionally, specimens with 25mm, 32mm and 46mm cover depth were tested under condition 2. The concrete had a cube crushing compressive strength of 35N/mm^2 . The aggregate was natural gravel with a maximum diameter of 19mm.

Temperatures up to 750°C were considered. Specimens were heated at a rate of 2°C per minute. After the target temperature was reached, the specimens were held at this temperature for a dwell period of one hour before being tested or cooled. The loading rate was equivalent to a bond stress increase of approximately 3N/mm² per minute.

Specimens were tested in batches of four within the purpose-built furnace. Load was applied to the end of each reinforcing bar which protruded out of the base of the furnace. Slip was measured from the top, unloaded end, of each specimen. The concrete compressive strength was measured at elevated temperatures to aid the analysis of the bond test results.

It was found that the bond strength reached a critical value where the concrete crushed immediately beneath the ribs. The change in bond strength with increasing temperature was found to follow the same basic pattern as that of compressive strength. The pattern was consistent between specimens tested in each of the four conditions listed. The reduction in bond strength was dependent on concrete compressive strength, and the decrease in bond strength was greater than that of compressive strength. Bond strength retention with increasing temperature under test condition 3 is shown in figure 2.33.

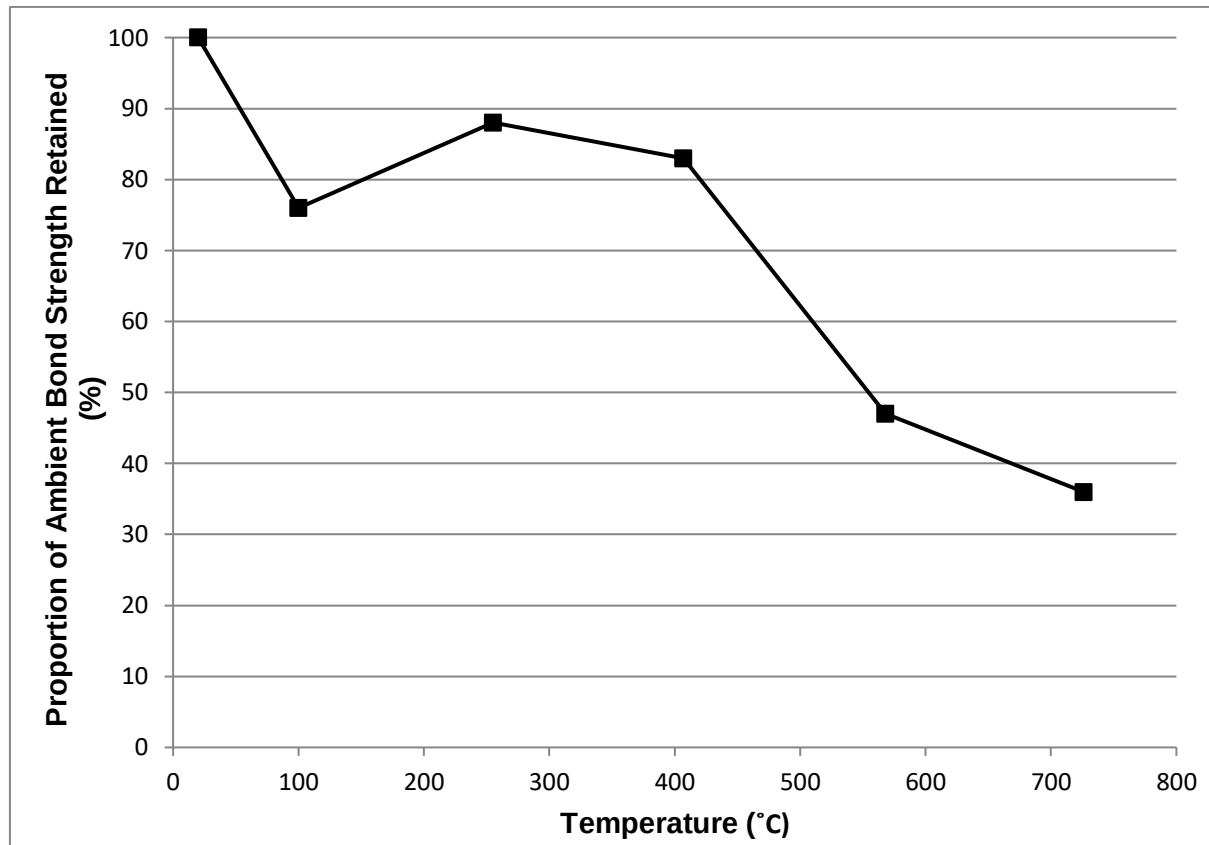


Figure 2.33. Proportion of bond strength retained at elevated temperatures for deformed bar under test condition 3 (Morley and Royles, 1983)

It was found that specimens which had been stressed during heating had slightly better performance at elevated temperatures than those that were not stressed. Specimens with smaller cover depths resulted in a tensile splitting failure mode while specimens with larger cover depths presented a pull-through failure mode with the concrete crushing below the ribs.

The trend in bond strength retention with increasing temperature was similar for plain and deformed bars, as can be seen in figure 2.34. It is important to note that this is a comparison of residual strength after cooling.

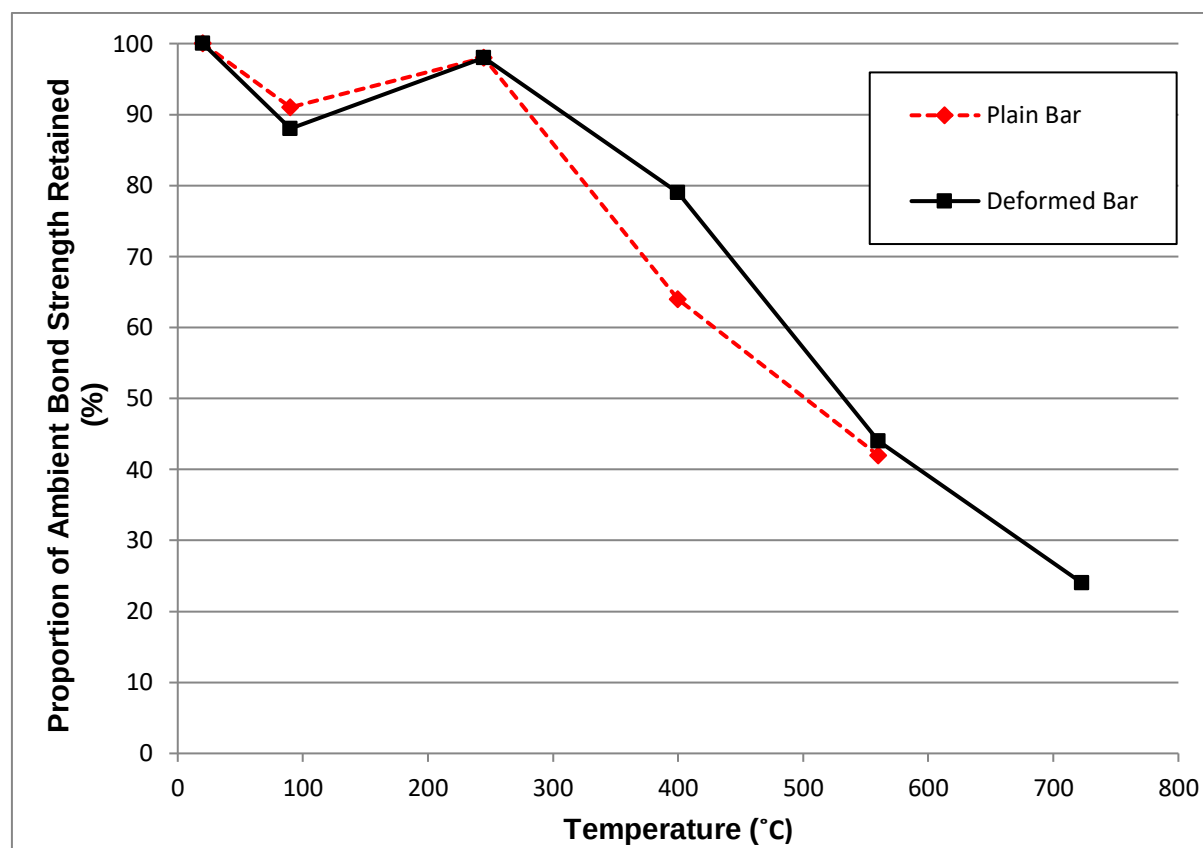


Figure 2.34. Proportion of bond strength retained at elevated temperatures for plain and deformed bar after cooling (test condition 2) (Morley and Royles, 1983)

2.8.1.2 Hertz

Hertz (1982) conducted elevated temperature pull-out experiments on 280 specimens. The specimens were conical shaped and were pulled against a conical shaped bearing block. The geometry of the test specimen and bearing block were as shown in figure 2.35.

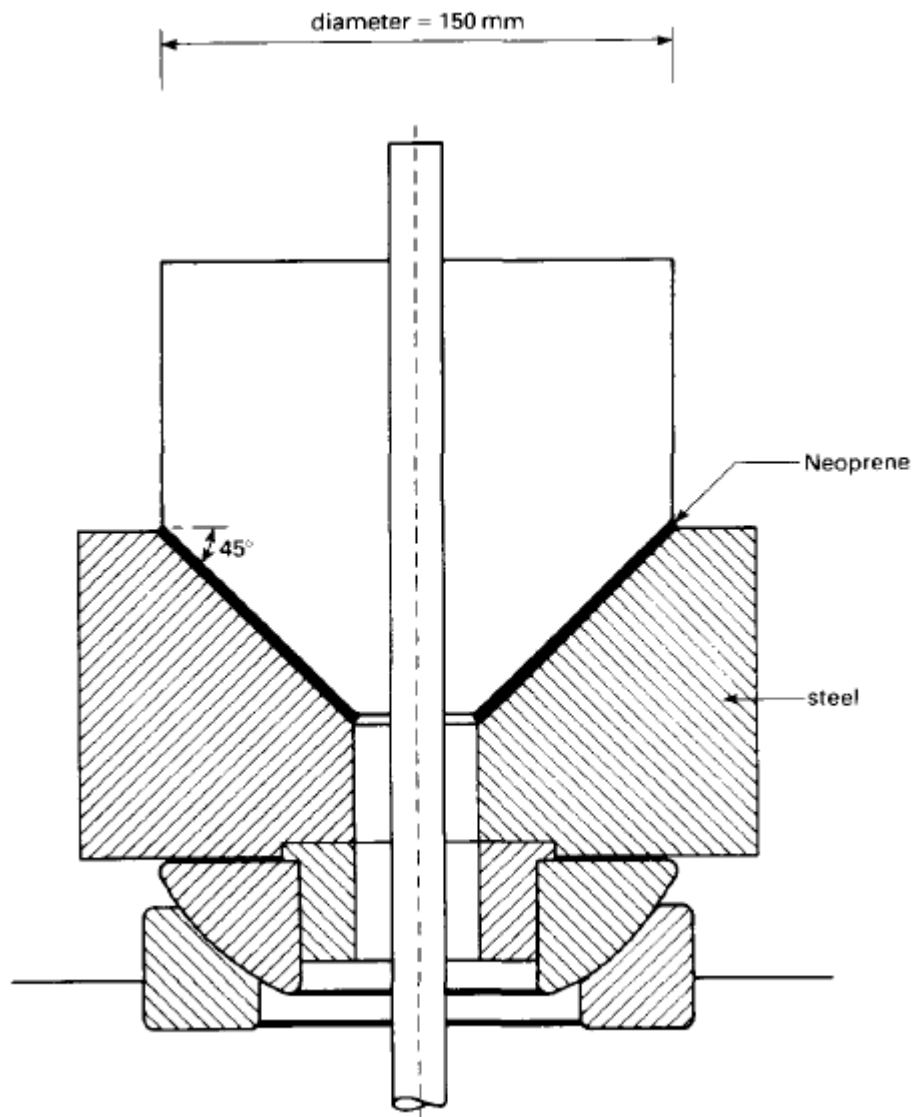


Figure 2.35. Pull-out test geometry from Hertz (1982)

Each specimen contained a single reinforcing bar. Reinforcing bars were either 8,12,16 or 25mm diameter deformed bars or 8,12 or 16mm diameter plain bars. The surface geometry of the deformed bars was different to that of typical deformed reinforcing bars currently being used in the UK. The concrete compressive strength was 20N/mm² which is quite weak in comparison to typical concrete in modern UK construction. The concrete aggregates were a Danish sea gravel, which was a mixture of quartz, granite and limestone.

The specimens were heated to target temperatures up to 800°C. The heating rate was 1°C per minute and the maximum temperature was held for a dwell period of 2 hours. The specimens were then cooled at a rate of no more than 1°C per minute.

Each specimen was loaded in order to determine the maximum force that could be transmitted by the bond after cooling from each temperature. The variation in bond strength retention with increasing temperature for 12mm diameter deformed Danish 'Tentor' bars is shown in figure 2.36. Deformed bars of other diameters performed similarly to the 12mm diameter bar in terms of bond strength retention at elevated temperatures.

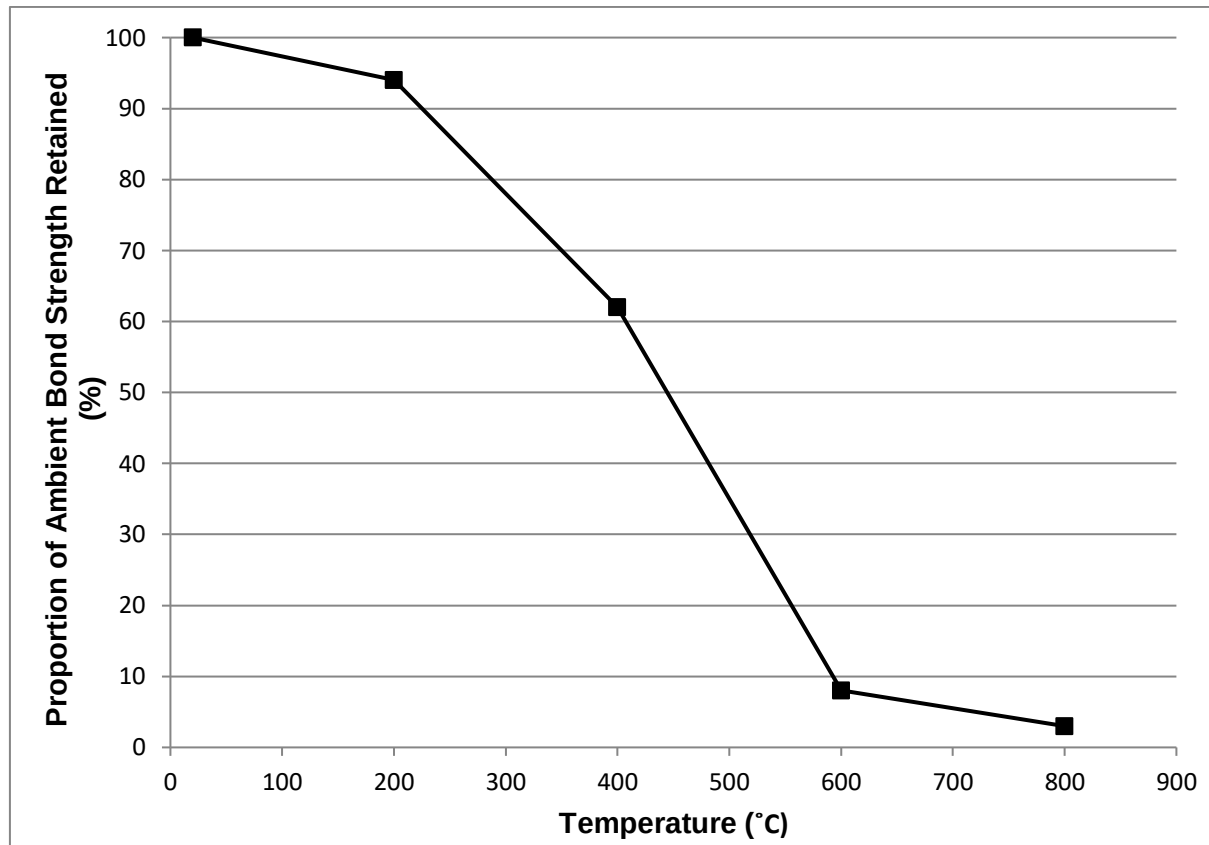


Figure 2.36. Bond strength retention of 12mm deformed bar at elevated temperatures (Hertz, 1982)

The bond strength retention of plain bars can be seen in figure 2.37. The decrease in bond strength of plain bars with increasing temperature was generally more severe than the decrease in deformed bars. It is important in the context of the current work to consider that these tests were conducted after cooling of specimens.

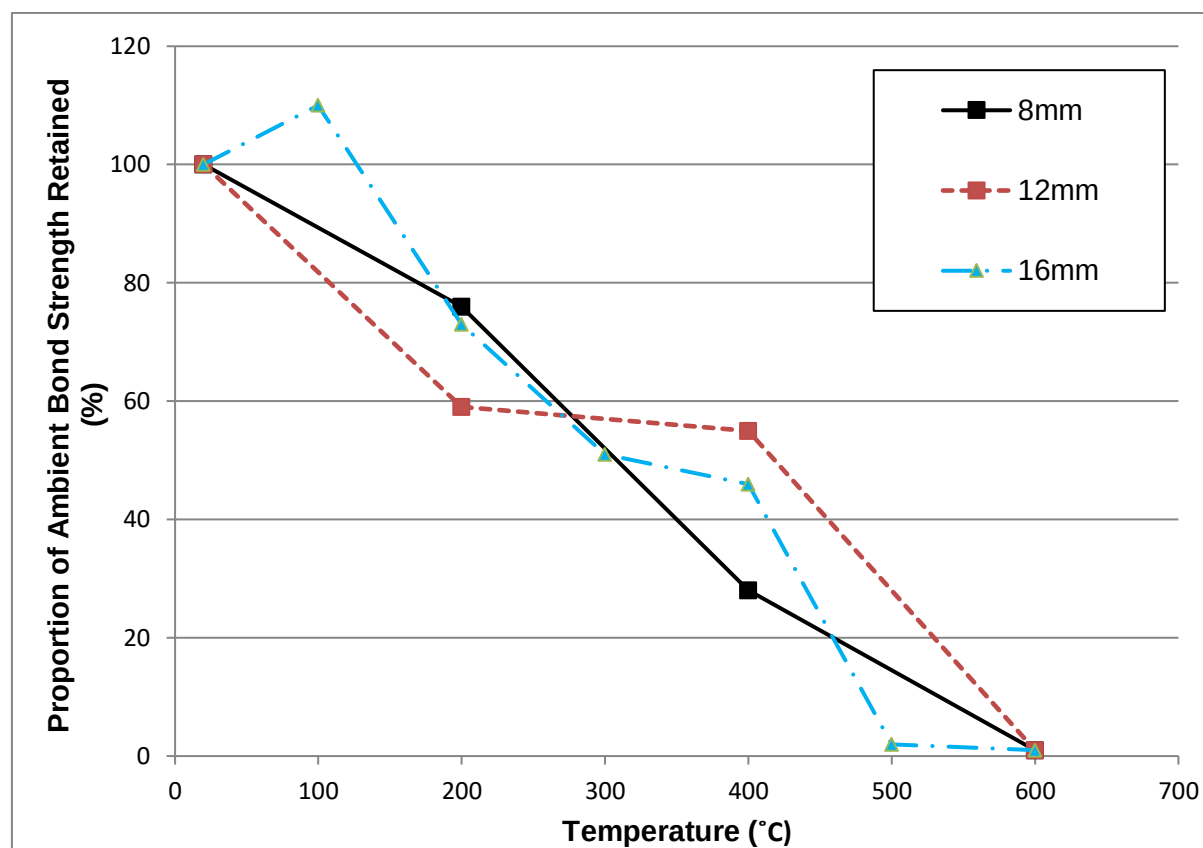


Figure 2.37. Bond strength retention of 8,12 and 16mm plain bars at elevated temperatures (Hertz, 1982)

2.8.1.3 Diederichs and Schneider

Diederichs and Schneider (1981,1982) performed elevated temperature pull-out tests. These tests were conducted to investigate the stress-slip characteristics of the bond. Parameters considered in this study included steel type, bar surface type and concrete types with different aggregates. The test arrangement is shown in Figure 2.38.

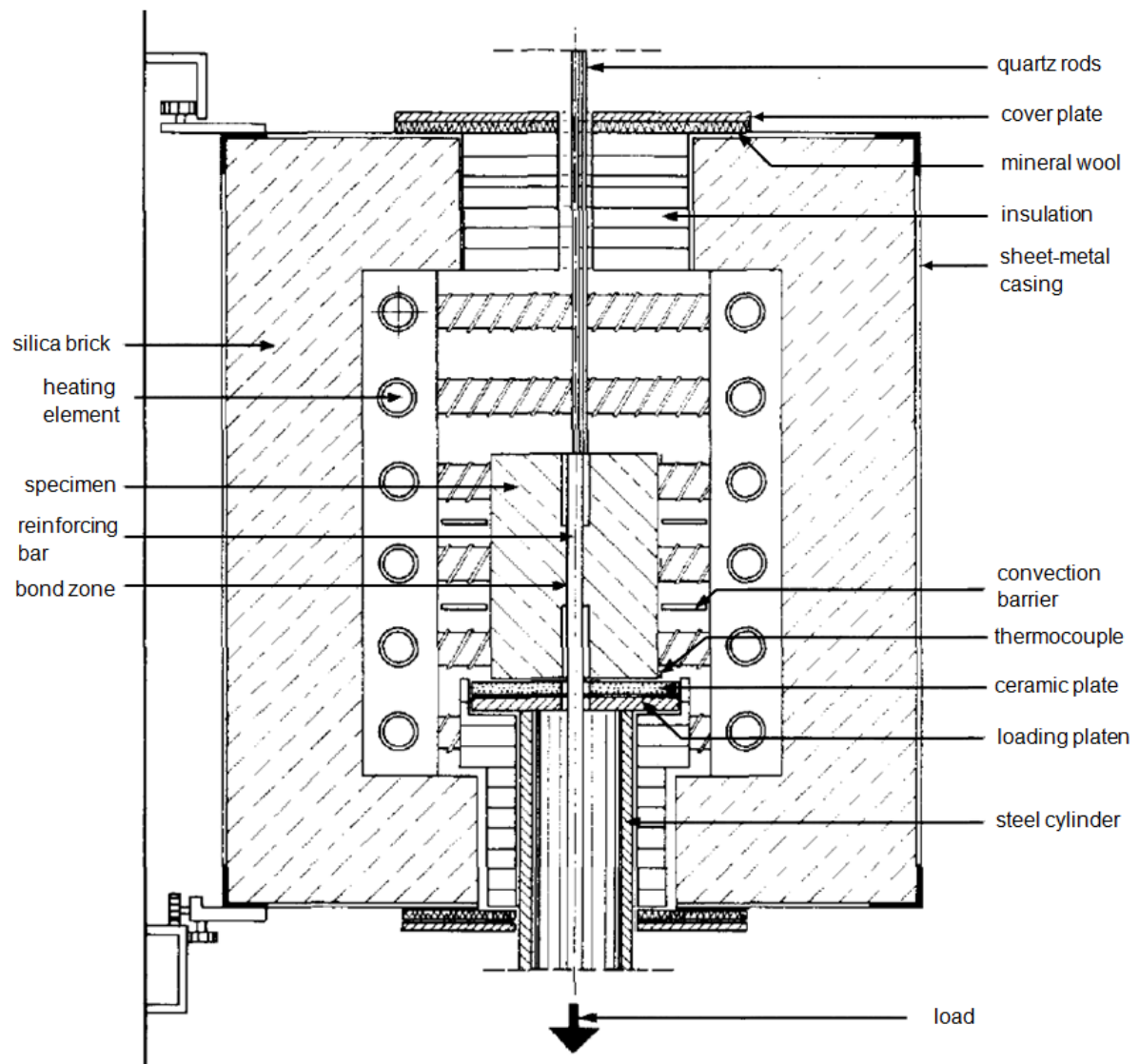


Figure 2.38. Test arrangement from Diederichs and Schneider (1981), drawing modified for clarity

The specimens were cylinders with 172mm diameter and 191mm height. Most specimens had an embedment length of five times bar diameter, but different bond lengths were considered, including 40mm and 80mm. Each specimen was made from concrete containing either siliceous gravel aggregate or crushed basalt aggregate. The concrete cube strength was measured with maximum and minimum values of 60.9N/mm² and 48N/mm² from 10 samples. Reinforcement bars tested included plain round bars and deformed bars.

Specimens were tested in one of the following two conditions:

1. force-slip measured at steady state high temperature
2. force-slip measured at variable, transient temperature condition

Condition 1 involved heating the specimen at 1°C per minute to the desired temperature and then holding the temperature steady for a three hour dwell period. The specimen was then loaded to failure at a rate of about 1kN/s. Condition 2 involved heating the specimen until failure with a certain load already applied.

The bond strength of plain bars decreased with increasing elevated temperatures. For specimens made with siliceous gravel aggregate, the adhesion decreased with increasing temperatures and disappeared for a 16mm plain bar at about 300°C and for an 8mm plain bar at about 400°C. However, for specimens made with basalt aggregate, some adhesion remained at temperatures up to 800°C.

Deformed bars had better bond strength retention at elevated temperatures than plain bars. The loss in bond strength of deformed bars and the loss in concrete compressive strength were of the same order of magnitude. The variation in bond strength with increasing temperature for each bar type is shown in figure 2.39.

With siliceous aggregate specimens the maximum bond stress was lower for an 8mm deformed bar than a 16mm deformed bar below 500°C, but greater at temperatures above 500°C. For specimens with basalt aggregate, the maximum bond stress was lower for 8mm deformed bar than for 16mm deformed bar irrespective of temperature.

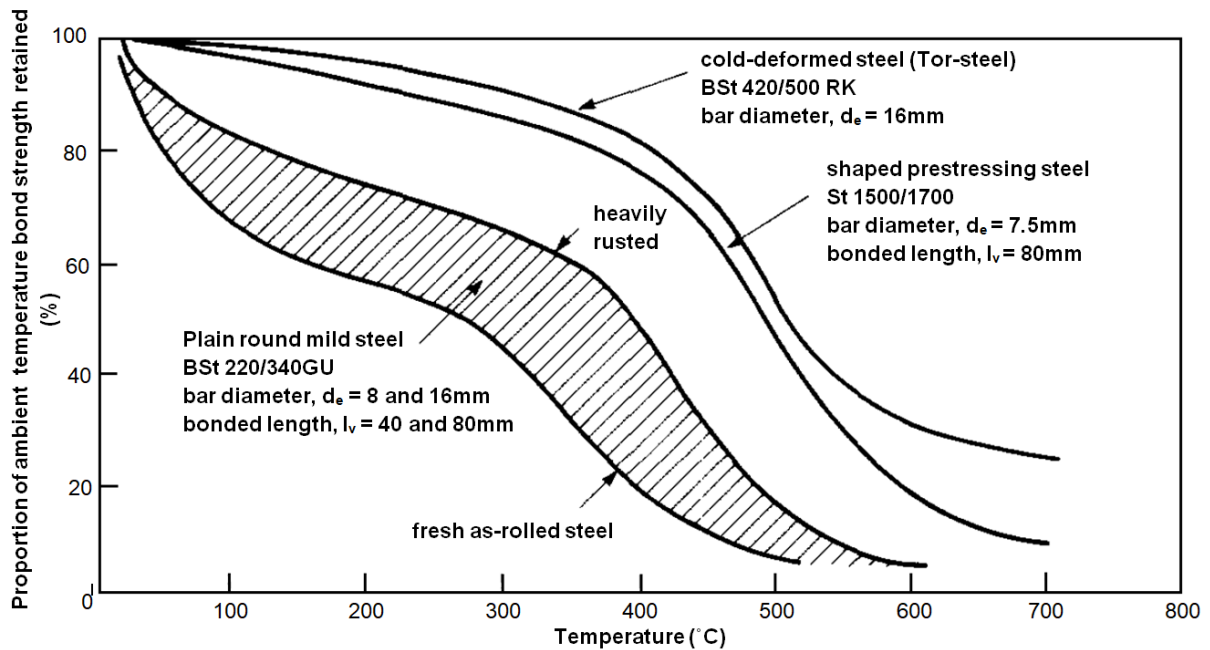


Figure 2.39. Relative bond strength retention of reinforcing bars with increasing temperature, modified from Diederichs and Schneider (1981)

The results in figure 2.39 show smooth curves with bond strength of decreasing bond strength with increased temperature up to 200°C for rusted plain round bar. The stress slip curves in figure 2.40, however, show a very modest increase in bond strength at 80°C and 200°C relative to ambient temperature. This increase was not explained, but it was mentioned in the discussion of these results that there was little difference in the stress-slip curves for rusted plain bars up to 200°C. The results in figures 2.39 appear to contradict those in figure 2.40 for rusted plain round bars up to 200°C.

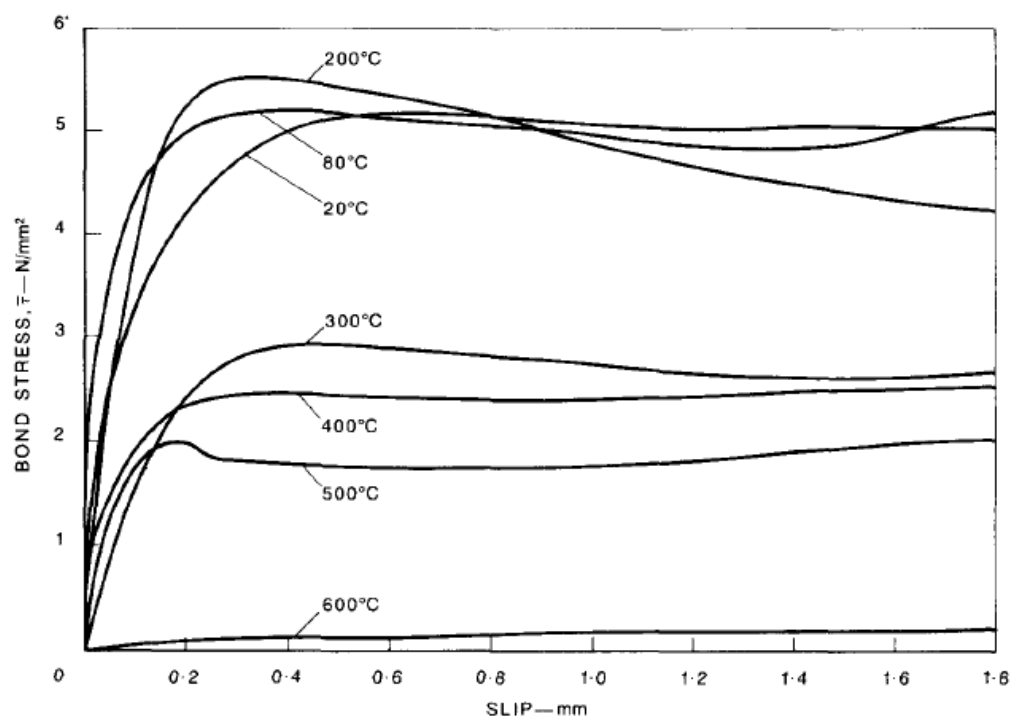


Figure 2.40. Bond stress-slip relationship for rusted, 8mm diameter, plain round bars at elevated temperatures, from Diederichs and Schneider (1981)

2.8.1.4 Giroldo and Bailey

Giroldo and Bailey (2008) conducted pull-out experiments to investigate the bond behaviour of welded mesh reinforcement at elevated temperatures. These experiments involved pulling the longitudinal bar which was embedded in concrete and welded to a transverse bar which was also embedded in the concrete. Bar diameters of 6,7,8 and 10mm were used in the study and both deformed and plain round bars were included (Giroldo, 2010). The longitudinal bars had a bonded length of 200mm, corresponding to the bar spacing of standard UK reinforcing mesh. The specimen geometry is shown in figure 2.41.

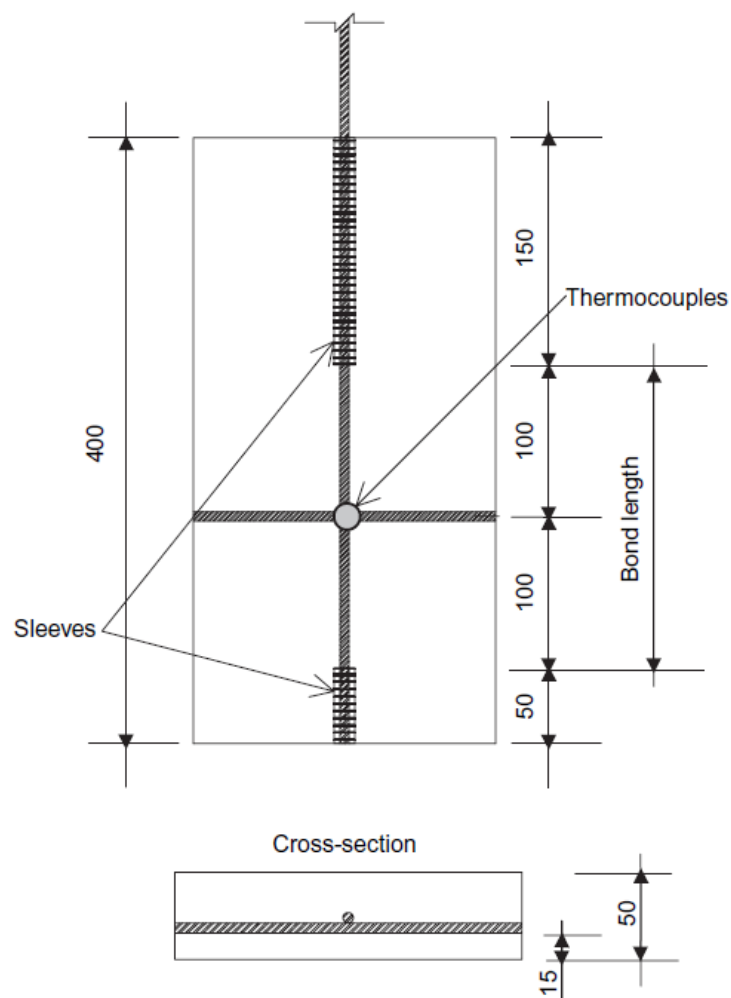


Figure 2.41. Test specimens from Giroldo and Bailey (2008)

The concrete had a characteristic compressive strength of 35N/mm^2 at 28 days. The aggregate was a natural gravel aggregate with a 10mm maximum size. The specimens were cast simultaneously with the same mix.

The tests were mostly conducted in steady-state temperature condition, although eight transient temperature tests were also performed. The steady-state tests were heated to the target temperature and then loaded to failure. The transient temperature tests were under a specific load prior to heating, and then heated at $300^\circ\text{C}/\text{hour}$ until failure. The steady-state test temperatures were 20, 200, 400, 600, 800 and 1000°C . The load applied in the transient tests was based on the results of the steady-state temperature tests and corresponded to the failure load at 400°C in the steady-state temperature condition.

The purpose of the transient tests was to identify the temperature at which failure occurred and compare this against the results from the steady-state tests to determine if there was any significant difference in the results. The results showed that there was not much difference in the results of steady-state and transient tests in terms of temperature at failure with the same failure load.

The plain bars all failed by fracture of the weld and subsequent pull-out of the bar. The 6,7 and 8mm deformed bars all failed by fracture of the reinforcement. Failure of the 10mm deformed bar specimen was by concrete splitting. There was no clear relationship between bar diameter and bond strength retention for the plain bars. Plain bars were found to have a greater decrease in bond strength with increasing temperature than deformed bars. The reduction in bond strength with increasing temperatures for deformed and plain bars is shown in figures 2.42 and 2.43 respectively.

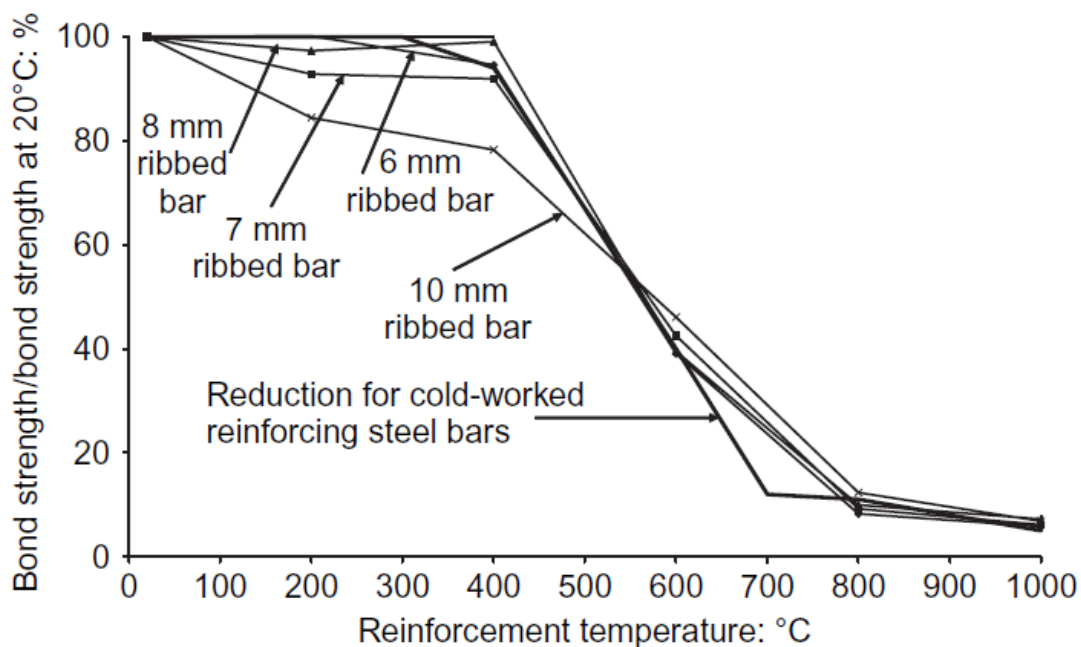


Figure 2.42. Reduction of deformed bar bond strength with temperature, from
Girollo and Bailey (2008)

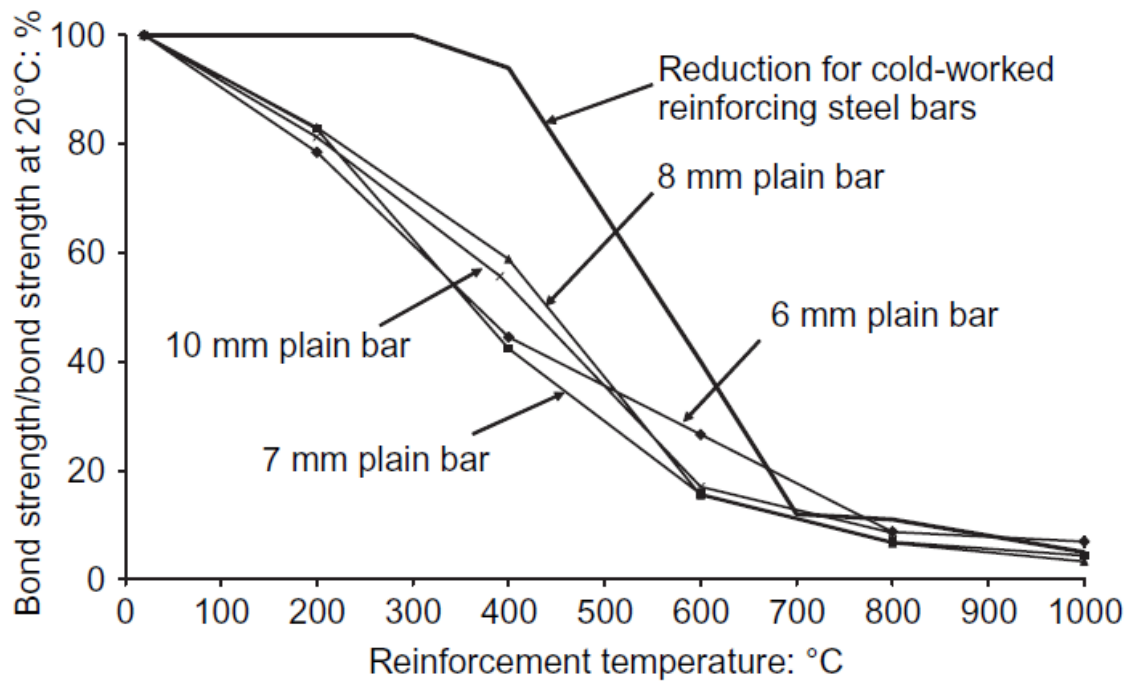


Figure 2.43. Reduction of plain bar bond strength with temperature, from Girollo and Bailey (2008)

It was argued that plain round bars would be more beneficial in composite floor slabs with full-depth cracks and reinforcement rupture. The justification given was that the bond of the plain round bars would not be strong enough to prevent loss of bar anchorage, and this would result in sufficient bond slip that the strains would be more distributed along the bar, making the bar less prone to rupture. It was suggested that deformed bars would have more localisation of strain at crack locations due to their bond performance, resulting in reinforcement fracture. It should be noted however, that the anchorage length of the plain bars in this study was only 200mm and, therefore, it was not confirmed whether the bond of a plain bar would actually fail if the anchorage length was longer. These tests, therefore, do not prove that either bar surface type would be more beneficial than the other in the scenario of a heavily cracked composite slab.

2.9 Summary of Chapter 2

It is evident from building fires and full scale testing that tensile membrane action can enhance the load bearing capacity of two way spanning slabs. Previous research has also demonstrated that in the case of slabs with horizontal and rotational support at the edges there is a greater enhancement of load bearing capacity than in unrestrained slabs, and that the failure mode can be expected to be rupture of crack-spanning reinforcement around the edges of the slabs.

Current design methods which take account of tensile membrane action for two way spanning slabs are based on experimental observations in which plain round bars were used. These design methods may be unconservative for slabs which are reinforced with modern deformed bars, as differences in bond and material properties between bar types may change the behaviour of reinforcement at crack locations around the edges of slabs acting in tensile membrane action. It is important to verify whether current design methods, which are based on observations of slabs reinforced with plain round bars, are appropriate for use with slabs reinforced with modern deformed reinforcement bars.

Pull out experiments provide an effective method with which to measure the effect of different conditions such as temperature or bar diameter on the bond stress-slip response. The bond strength of deformed reinforcing bars is mostly provided by mechanical interlock, and the bond strength of plain round bars is mostly due to chemical adhesion. Concrete compressive strength follows a down-up-down trend with increasing temperature. Relative to siliceous aggregate concrete, the thermal expansivity of steel is greater up to around 450°C and lower at higher temperatures.

Previous elevated temperature bond tests have identified that bond strength of deformed and plain round bars generally decreases with increasing temperature.

As technical specifications of bar surface geometry and mechanical properties have changed in recent decades, it is not known how modern deformed reinforcing materials will be affected by elevated temperatures as previous studies were on older reinforcing materials. There is a particular lack of knowledge of the effects of elevated temperature on the stiffness of the bond stress-slip response. Accurately modelling the rupture of crack-spanning modern deformed reinforcement at elevated temperatures will require the bond stress-slip response of modern deformed reinforcement at elevated temperatures to be characterised.

3 Experimental programme on elevated temperature bond

3.1 Experimental design and methodology

This chapter describes a series of experiments which were conducted to investigate the effect of elevated temperature on the bond behaviour of modern steel reinforcement in concrete. This research is required as it is unknown how the bond of modern reinforcing bars and concrete will behave at elevated temperatures. It is also useful to test plain bars in the same series of tests as deformed bars so that comparisons can be made of the relative performance of these bar types. The results will be useful for further research and the stress-slip results were used in the numerical modelling described in later chapters of this research.

Elevated temperature pull-out tests have been conducted in the engineering laboratories at the University of Edinburgh. An annotated diagram of the test set up is shown in figure 3.1. This test setup will be explained in the following sections. The key test parameters are shown in table 3.1. Four different reinforcing bars have been considered, these are plain round bars of 8mm diameter and deformed bars of 8, 10 and 12mm diameter. These bar diameters were selected as they are commonly used in construction and they are small enough to fit inside the grips of the universal test machine which was used.

Type of bar	Bar diameter	Target test temperatures
Plain round	8mm	Ambient, 110°C, 210°C, 300°C and 600°C
Deformed	8mm	
	10mm	
	12mm	

Table 3.1. Key test parameters

The four bars have been tested at five different temperatures resulting in twenty different test conditions. The test temperatures, and their deviation between tests, are listed in table 3.4. Narrower temperature intervals were chosen at the lower end of this range due to the more complex non-monotonic trends which concrete and bond strength exhibit at these temperatures.

This range of temperatures was considered to be the most relevant for design based on relevant design codes. Eurocode 4 Part 1-2 (BSI, 2005b) gives conservative estimates of temperature distribution in a 100mm thick solid slab of normal weight concrete after exposure to different durations of the ISO standard fire. Based on these conservative estimates, 30mm of concrete cover would be sufficient to keep a reinforcing bar just below 600°C after 120 minutes exposure to standard fire.

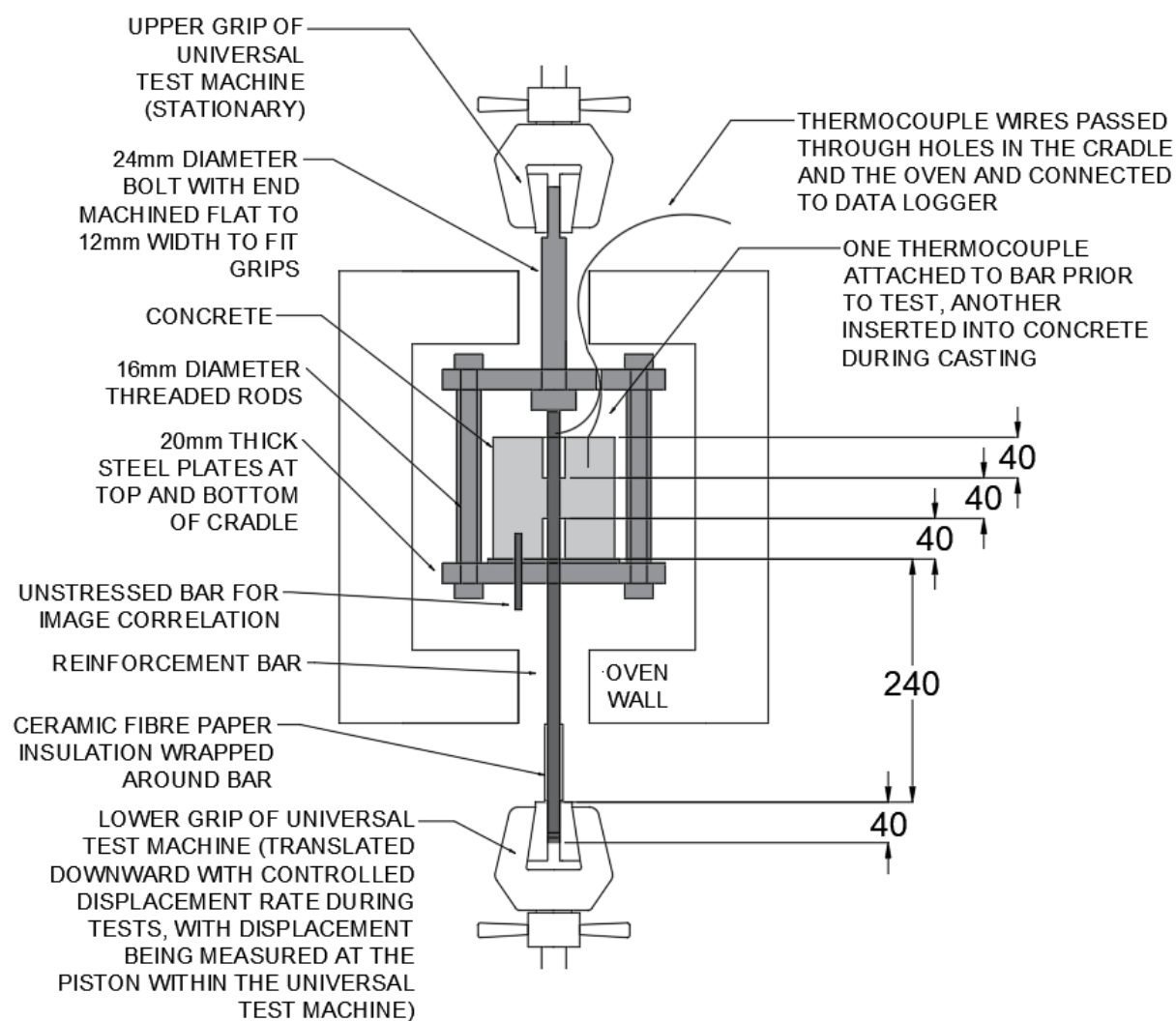


Figure 3.1. Annotated diagram of test setup, with dimensions in millimetres

Table 3.2 lists all of the tests which were conducted. This table also includes some comments describing any issues with the tests. These issues are addressed in section 3.2.2.

Type of bar	Bar diameter (mm)	Test target temperature (°C)	Number of tests performed	Comments on tests (further descriptions follow in this chapter)
Plain	8	Ambient	4	
		110	3	
		210	3	
		300	3	
		600	3	
Deformed	8	Ambient	4	
		110	3	
		210	3	One test had unintentional pre-load
		300	3	
		600	3	
Deformed	10	Ambient	4	One test exhibited splitting failure (see section 3.2.2)
		110	3	
		210	3	
		300	3	
		600	3	
Deformed	12	Ambient	4	One test removed as outlier
		110	4	Two tests exhibited splitting failure
		210	4	Two tests exhibited splitting failure
		300	3	One test had unintentional pre-load (see section 3.2.2)
		600	3	

Table 3.2. List of pull-out tests conducted

3.1.1 Specimen geometry

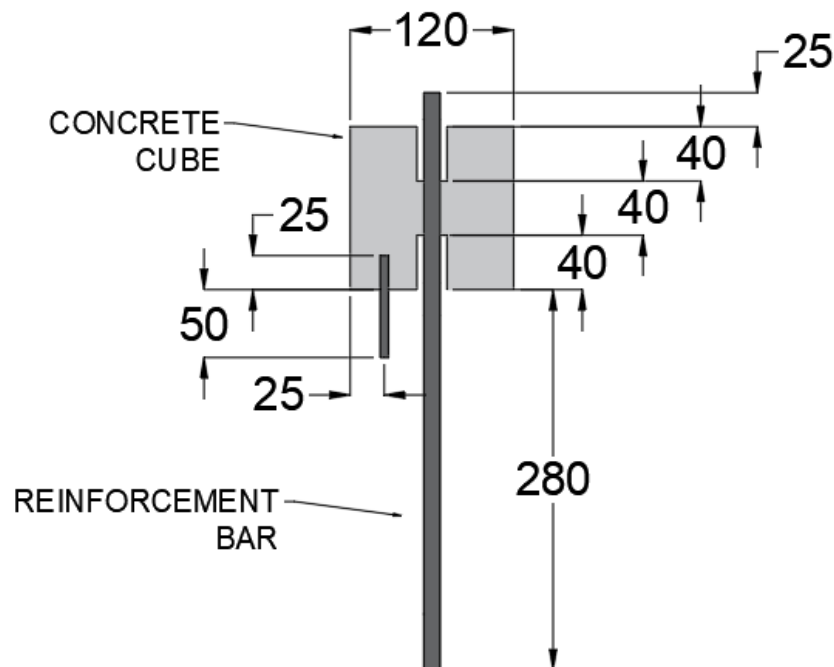


Figure 3.2. Long section of experimental specimen with dimensions in millimetres

The specimen geometry is shown in figure 3.2. The experimental specimens were 120mm wide concrete cubes with a centrally embedded reinforcing bar. Only the middle 40mm of the embedded length of the bar was bonded and in contact with the concrete. The 40mm bonded length is five times the minimum bar diameter: the conventional ratio for such tests (Cairns, 2020). The geometry was intended to be simple and to have sufficient cover that splitting would be avoided. Trial runs demonstrated that this was a suitable specimen size which would not be prone to splitting failures. The reinforcing bars were each 425mm long, with 25mm protruding from the top. Figure 3.3 shows four specimens with each of the four bar types. A short 6mm diameter plain bar was included, as shown to the left of the pull-out bar in figure 3.2. No loading or displacement was applied to this shorter bar. The shorter bar was intended to be used as a reference point for digital image correlation to track

displacement of the stressed reinforcing bar. This digital image correlation was not fully performed, but the bar was still used as a reference point as explained in section 3.2.1.

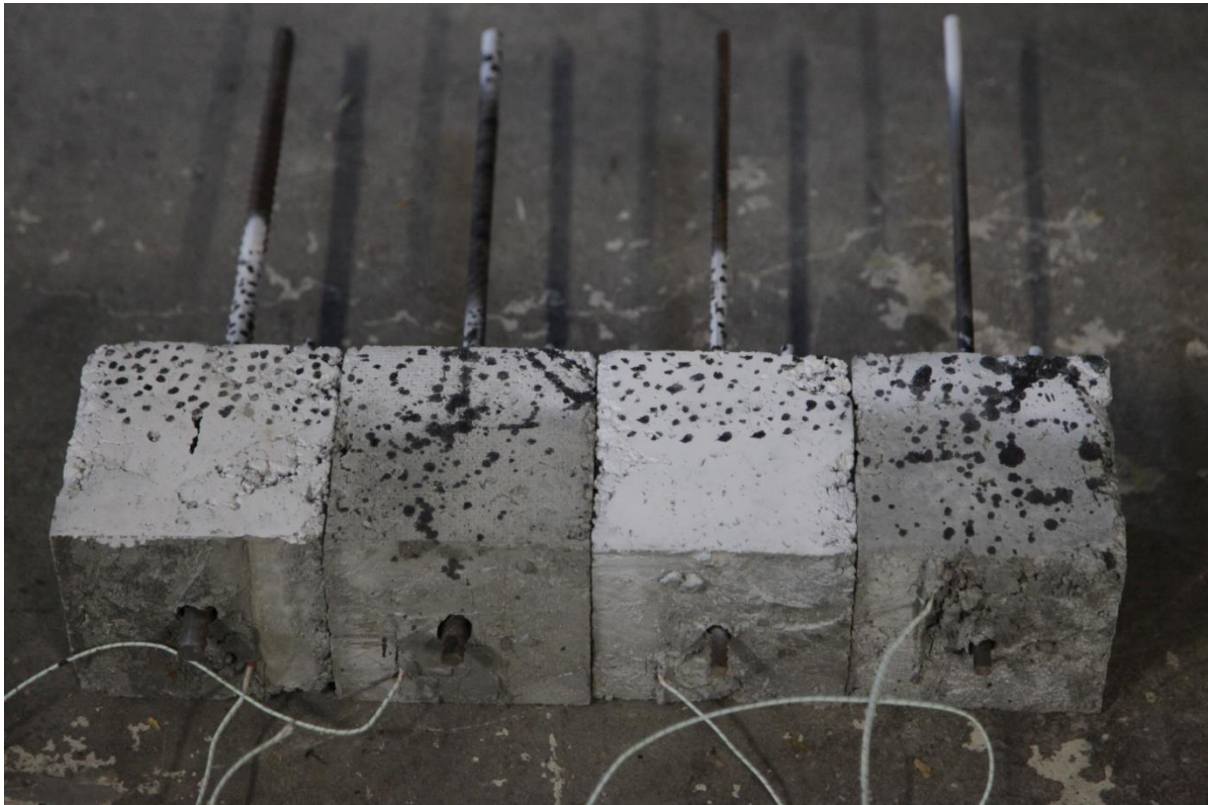


Figure 3.3. Photograph of specimens containing 12mm, 10mm and 8mm deformed bars and 8mm plain round bars (shown in this order from left to right), with thermocouple wires visible in the foreground

Each specimen had thermocouples included at the locations shown in figure 3.4. One thermocouple was attached to the bar, immediately above the top surface of the cube in order to measure the temperature near to the bond without interfering with the bond. Another thermocouple was inserted into the concrete in order to measure temperature at a location which was chosen as it is far enough inside the concrete to provide internal temperatures in a well insulated region and far enough from the bar

that measurements give an indication the size of the temperature differential from the bar and to avoid interfering with the bond.

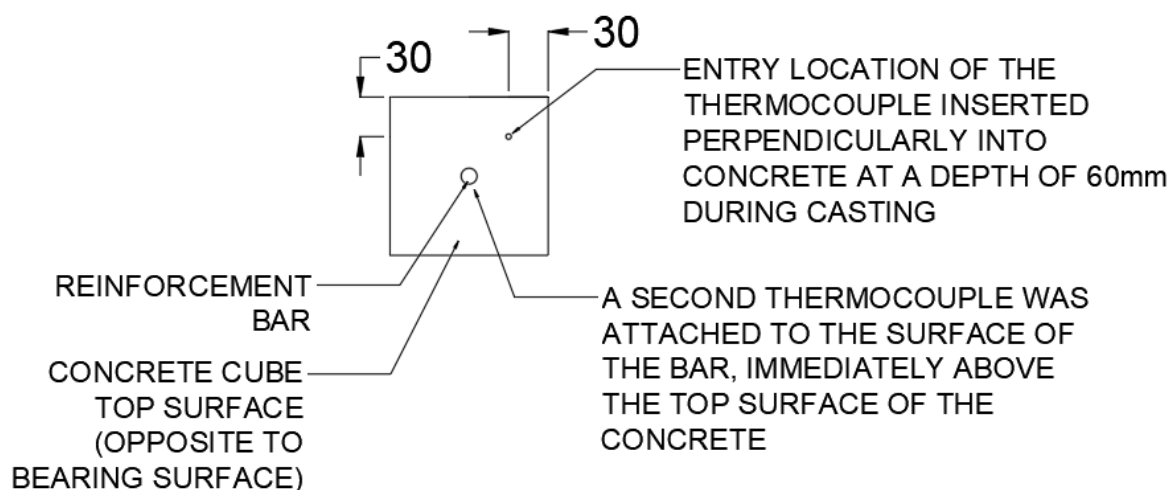


Figure 3.4. Drawing showing positions of thermocouples, with dimensions in millimetres

3.1.2 Specimen materials

3.1.2.1 Concrete

The concrete mix design followed the procedures given in the Building Research Establishment document 'Design of Normal Concrete Mixes' (Teychenné *et al.*, 1997). The cement was ordinary Portland cement. The aggregate was a siliceous gravel with 10mm maximum size. The concrete was designed with a target characteristic strength of 35N/mm^2 at 28 days. The risk of spalling at elevated temperatures was reduced by an extended curing period. The specimens were all made from a single batch of concrete. The specimens were cured in laboratory conditions, with ordinary air cooling and at temperatures of approximately 15 to 20°C . The curing conditions were consistent for all specimens.

The pull-out tests were interrupted by equipment damage and by laboratory closure due to the COVID-19 pandemic. This resulted in intervals between batches of tests. All except three of the pull-out tests were conducted within a 32 day period commencing 272 days after casting or a 12 day period commencing 495 days after casting, as shown in table 3.3.

The final pull-out test was conducted 506 days after casting. At 506 days after casting, the average compressive cube strength of the concrete from 100mm cubes was measured as 53.4N/mm^2 . At 506 days, the tensile strength of the concrete was measured as 3.5N/mm^2 from splitting tests on concrete cylinders of 300mm length and 150mm diameter, sized to technical standard BS EN 12390-6:2009 (BSI, 2010), although loading rate was not perfectly consistent due to the use of a manually controlled test machine. Concrete compressive strength was estimated at different ages using a mathematical model proposed by Abd Elaty (2014) to predict strength

gain of Portland cement concrete with reasonable accuracy. These estimated compressive strengths are shown in table 3.3. The differences in concrete strength are considered in the interpretation of the results.

Bar Description	Test Target Temperature (°C)	Quantity	Curing Period (Days)	Compressive Strength (N/mm²)
8mm deformed bar	15	1	82 -90	41.8 (estimated)
12mm deformed bar	15	3*		
8mm plain bar	300	3	272 - 303	49.7 (estimated)
8mm deformed bar	300	3		
8mm deformed bar	600	3		
10mm deformed bar	300	2		
10mm deformed bar	600	3		
12mm deformed bar	300	3		
12mm deformed bar	600	3		
Remaining experiments	various	43	495-506	53.4 (measured)
(*one of these three tests was the outlier, mentioned in table 3.3, which was excluded from further analysis				

Table 3.3. Specimen curing times and estimated compressive strengths

Due to shorter curing periods, the average compressive strength of the specimens tested at ambient temperature will be lower than those tested at elevated temperature. This is most severe in the 12mm deformed bar group of specimens, where the average compressive strength of the ambient specimens was approximately 12% lower than the average compressive strength of specimens tested at elevated temperatures, based on the estimated strengths used in table 3.3. This will have some influence on the results, and given that the bond of deformed bars can be expected to be stronger with higher strength concrete, it can be expected that the results will show an unrepresentatively high degree of bond strength retention at elevated temperature for the 12mm deformed bar. This effect will be less significant for 8 and 10mm diameter bar specimens as the differences in compressive strength of specimens was, on average, smaller.

The effect of differences in concrete compressive strength could not be corrected for in the results because the effect could not be quantified with confidence, given that there are other factors beyond concrete compressive strength which influence bond strength at elevated temperature. As this effect has not been corrected for in the results, it is important to consider that the analysis of the results will be less reliable, particularly for the trend in bond strength retention of the 12mm deformed bar at elevated temperature.

3.1.2.2 Steel

The 8mm diameter plain round steel bars were UK grade S275 to technical standard BS EN 10025-1:2004 (BSI, 2004), with a specified yield strength of 275N/mm². Historical plain round reinforcing bars were typically of the older 'grade 250' (BSI, 1978) which had a yield strength of 250N/mm². This material could not be sourced and since the bars would not reach yield in these tests there will be no significant effect on the results. The plain round bars had a mill finish with light corrosion to the surface. It is common for reinforcement to have some corrosion in practice, and surface roughness of corrosion will increase bond strength. Figure 3.5 shows the condition of one of the plain bars along with one of each diameter of the deformed bars. The bars used in the specimens were all of similar condition.



Figure 3.5. Plain bar and 8, 10 and 12mm diameter deformed bars (from top to bottom)

The deformed bars were typical of modern construction in the UK. These bars were UK grade 500B to technical standard BS4449:2005 (BSI, 2005c). This specification of reinforcing bar has a minimum yield strength of 500N/mm² and a total elongation

of 5% at maximum force. The rib pattern was similar for each bar diameter and is visible in figure 3.5. The rib spacing was 5mm for the 8mm bar and 7mm for the 10 and 12mm bars. There was also some light surface corrosion on the deformed bars, and it is expected that this would be less significant for deformed bars given that the contribution to mechanical interlock provided by the ribs is far greater than the contribution of surface roughness. As with plain bars, it is common for the bars to have some surface corrosion when used in construction.

3.1.3 Casting

The reinforcement bars were covered with two short lengths of hose pipe in order to limit the bonded length, as shown in figure 3.6. Different diameters of hose pipe were used for different bar types to ensure that they fitted tightly on each of the reinforcing bars. The hose pipe was fitted to measured locations along the bar length and each fitted tightly enough that there was no risk of it unintentionally slipping in the chosen casting method. The hose pipe was removed from each specimen 28 days after casting.



Figure 3.6. Photograph showing each bar type and diameter with hose pipe fitted in position to limit length of bonded region

Specimens were cast vertically and in the direction of bar pull-out. Although this casting direction means that the ribs of deformed bars may need to slip slightly before being brought fully into bearing, as explained in section 2.4.1, the specimens were all cast in the same direction and are, therefore, comparable. All specimens were cast from a single batch of concrete. This casting approach resulted in a smooth bearing surface on each cube. Purpose built moulds were used to cast the specimens. The moulds, shown in figure 3.7, were constructed from phenolic faced plywood. The moulds included holes to allow the reinforcing bar of each specimen to be inserted and secured in the correct position. Specimens were filled in three layers and compacted with a tamping rod, with 25 strokes per layer. Compacting by hand was preferred over vibration due to concerns that the reinforcing bars or fitted hose pipe could be moved by vibration.



Figure 3.7. Photograph showing casting moulds for specimens with reinforcing bars
in position

Figure 3.8 shows the specimens after casting with the thermocouples installed. Thermocouples were inserted into each specimen while the concrete was still workable. The position of the thermocouples was secured from above the specimens by fixing them to slots in strips of timber.



Figure 3.8. Photograph showing specimens after casting and insertion of thermocouple wires

3.1.4 Test setup

Pull-out tests were performed using a universal test machine with an attached oven. This arrangement can be seen in figures 3.1 and 3.9. A robust steel cradle with 20mm thick steel plates was designed by the author, and then fabricated for the tests. The cradle was located inside of the oven as shown in figure 3.10, with the cradle supported at the top by the upper tension grip of the universal test machine. The specimens were placed inside this cradle for testing. A steel mesh covered the open sides of the steel cradle as a barrier to stop any explosively spalled pieces of concrete from causing any harm, although no spalling was noticed during the tests. The reinforcing bar extended out through the bottom of the cradle and through a hole in the bottom of the oven. The bottom 40mm of the reinforcing bar was secured in the lower tension grip immediately prior to pull-out. The grip was not tightened prior to this stage as thermal expansion of the specimen could have damaged the grips.



Figure 3.9. Photograph showing oven between upper and lower grips of the universal test machine, with test specimen visible through the window



Figure 3.10. Photograph showing the steel cradle positioned inside the oven and being held from above by the upper grip of the universal test machine

The specimens were heated inside the electric oven shown in figures 3.9 and 3.10. A fan at the rear of the oven increased circulation of air inside the oven. The oven had a control system which allowed the heating rate and dwell period to be programmed and controlled automatically. The maximum operating temperature of the oven is 600°C, but this temperature could not be achieved due to some heat loss through holes which were left open for the passage of the reinforcing bar and thermocouples. The length of reinforcing bar between the oven and lower grip was insulated with two layers of ceramic fibre paper to reduce heat loss.

3.1.5 Instrumentation

As mentioned in section 3.1.1, thermocouples were inserted into the specimen during casting at the location shown in figure 3.4. An additional thermocouple was attached directly to the reinforcing bar at the point where it extended out from the top of the concrete cube. Thermocouples were type K and fibreglass insulated. The thermocouples ran through a hole in the top of the oven and connected to a data logger.

The universal test machine measured the applied force and displacement with respect to time during pull-out. The pull-out force values were converted into bond stress values by dividing the pull-out force by the area of the bond. Slip values were calculated from the measured displacements by subtracting the temperature dependent elastic deformation of the bar, as described in section 3.2.2.

A camera was positioned in front of the oven, as can be seen in figure 3.9, with a view of the bar exiting the bottom of the concrete cube. The photographs taken with this camera were intended to be for digital image correlation purposes. It was intended that digital image correlation would be used to verify the calculation of slip values. Due to being unable to make the digital image correlation program reliably recognise the surface pattern in the photographs, this system was not utilised. Instead of this, for one test, identifiable points on the surface of the bar were tracked, using CAD software, between photograph frames of equal time interval to verify the adequacy of the approach to calculating slip values. This process is described in section 3.2.1.

3.1.6 Test Procedure

3.1.6.1 Heating regime

With the specimen placed inside the oven, the gas temperature in the oven was increased at a rate of 5°C per minute until the target temperature was reached. The heating rate was chosen to avoid steep thermal gradients while allowing the tests to be completed within a working day. The gas temperature was then held constant for a specified dwell period to reach a more uniform temperature distribution throughout the specimen.

The dwell periods were chosen on the basis of balancing temperature uniformity and the feasibility of completing tests in a working day. A heat transfer analysis had incorrectly predicted a more uniform temperature distribution. The boundary conditions and interactions of the heat transfer analysis had been unrepresentative. The largest measured temperature differential between the thermocouples embedded in concrete and those attached to the steel bar occurred in the 570°C tests, where the average difference was 30°C during pull-out. The results will have been influenced by this temperature differential to an extent, but the effect will be limited since the critical region for a pull-out failure is very localised around the bar. Additionally, in an actual fire scenario there would be considerable temperature gradients present so this is no less representative of a real scenario than a uniform temperature distribution would have been.

Target temperature (°C)	Ambient	110	210	300	600
Measured temperature - average of specimens (°C)	15	107.4	206.6	291.4	573.4
Nominal temperature (°C)	15	110	210	290	570
Standard dev. of temperature between tests (°C)	<5	3.6	7.1	5.9	8.2
Dwell period (minutes)	N/A	60	90	110	120

Table 3.4. Test temperatures on steel bar adjacent to bonded region

The initial intention was for the tests to be conducted at 100, 200, 300 and 600°C. The target temperatures listed in table 3.4 refer to those which were defined in the heating regime and were set through the oven control settings. The specimens could not reach the target temperatures due to heat loss in the test setup. In an attempt to compensate for this, the target temperatures of 110°C and 210°C were used so that the actual temperatures of the specimens would reach at least the levels of 100°C and 200°C. The plan to compensate for heat loss was formulated after the 300°C tests had initiated, so the 300°C target temperature was not raised. It was not possible to raise the target temperature of the 600°C test as this was the maximum operating temperature of the oven. For clarity, the reason why the measured and nominal temperatures are lower than the target temperatures is heat loss in the test setup.

3.1.6.2 Loading

No load was applied during heating. After the dwell period had completed, the lower grip was tightened on the lower part of the reinforcing bar. The bar was then pulled by the lower grip of the universal test machine at a controlled displacement rate of 2mm per minute, as measured at the piston.

3.2 Results and analysis for elevated temperature bond experiments

Section 3.2.1 describes the approach taken to verify the calculated bond slip values. The results were processed to produce useful stress-slip curves. The steps involved in processing the results are described in section 3.2.2, along with other considerations in the interpretation of the results. Sections 3.2.3 to 3.2.5 present the results and analysis of the pull-out tests. The findings of these tests are discussed in section 3.3.

3.2.1 Verification of bond slip calculation

This section explains the verification of the method used to calculate slip by subtracting the elastic deformation from the displacements measured by the test machine. Photographs were taken with a digital camera, set up on a tripod, in front of the window of the oven. Photographs were taken automatically at 10 second intervals from the beginning of the test. An 8mm diameter test conducted at ambient temperature was used to verify the slip values which had been calculated from the measured displacements. The distance between identifiable points on the surface of the stressed reinforcement bar and an identifiable point on the unstressed bar in the images was measured using CAD software. The distances measured in the image were converted to real distances in millimetres based on measurements of a

photograph which was taken with a ruler placed alongside one of the reinforcing bars, as can be seen in figure 3.11. Figures 3.12 and 3.13 show close up views of the images for the 8mm diameter bar tested at ambient temperature at times of 30 seconds and 360 seconds respectively as examples of the images used for tracking. These images were included in this thesis solely to aid in the explanation of the method used to track displacements from the images.

Table 3.5 shows a comparison of the slip values calculated from the test machine displacements and those calculated from images. It was found that the displacements measured between images at higher displacements and time intervals correlated well with the slip values which were calculated from the displacements measured by the test machine. This demonstrates that the method of calculating slip by subtracting the temperature dependent elastic modulus was valid.

At low values of displacement and time, however, the slip values determined by these two methods were not in good agreement. This demonstrates that there was some settling in of the specimen and cradle at the beginning of the test, which would have distorted the calculated values of slip from displacement measured by the test machine. The effect of this settling in of the specimen and cradle was considered in the interpretation of the results, as discussed in section 3.2.2.

Measurement time		Increase in slip between time 1 and time 2		Difference in calculated slip values between methods (mm)
Time 1 (seconds)	Time 2 (seconds)	calculated from test measurement (mm)	calculated from images (mm)	
0	420	13.9	11.9	2
0	120	3.5	1.3	2.2
120	240	4.4	4.5	-0.1
240	360	4.1	4.1	0.0

Table 3.5. Comparison of slip values calculated from test machine measurements and those calculated from correlation of photographs



Figure 3.11. Photograph showing a ruler alongside the reinforcing bar, with the short bar used for reference visible in the bottom left of the image.

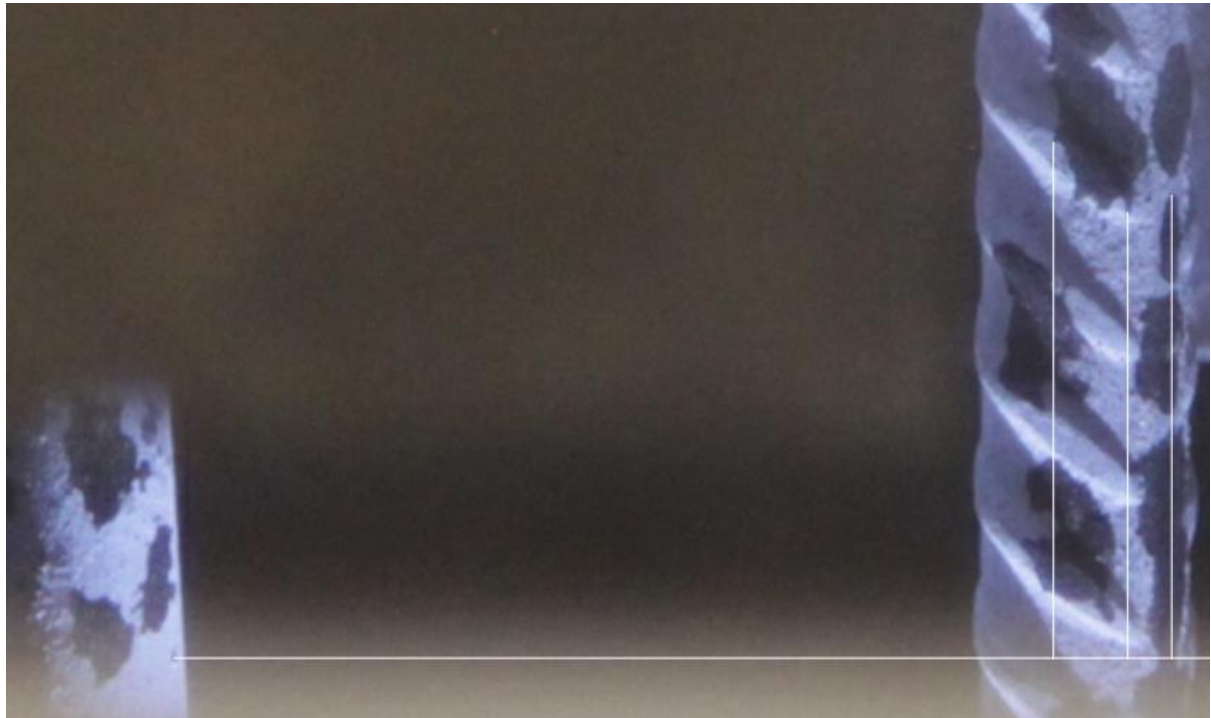


Figure 3.12. Photograph of 8mm deformed bar tested at ambient temperature, at 30 seconds after commencing pull-out, with markings used to measure relative position

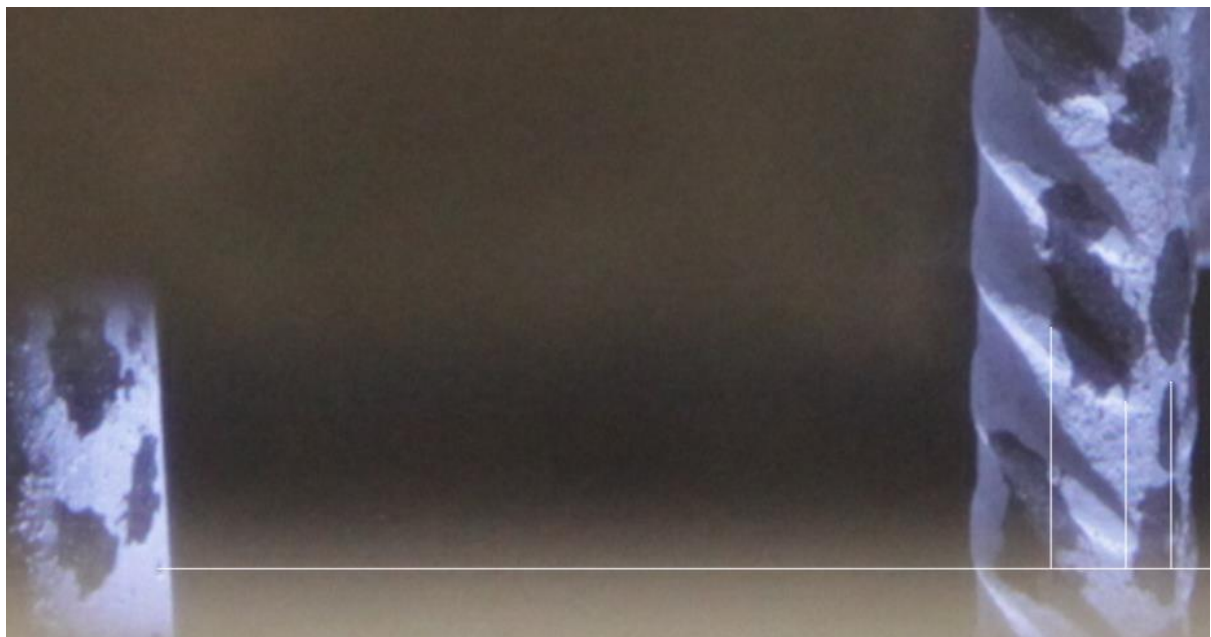


Figure 3.13. Photograph of 8mm deformed bar tested at ambient temperature, at 360 seconds after commencing pull-out, with markings used to measure relative position

3.2.2 Interpretation of the results

The results presented are mean values for each bar and temperature considered. Splitting failures have been excluded, and one further test which did not exhibit splitting has been excluded as an outlier. The splitting failure tests were from sets which were tested in quadruplicate, as shown in table 3.2, but in two of these instances the data points are averaged from only two tests, after splitting failures were excluded, which will increase the margin of error in those particular data points. This is significant for those individual data points, but is considered to be of less relevance in analysing the overall trends as they are formed by a larger number of tests. The bond strength of the outlier was less than one fifth of the other specimens which were tested under that condition and it is expected that the specimen was either damaged or incorrectly cast. The outlier was one of four tests for that bar diameter and temperature, so the results for that condition are still in triplicate after exclusion of the outlier.

The load measurement for two of the tests, listed in table 3.2, appeared to be incorrectly zeroed due to an unintentional pre-load. The values were corrected to include the pre-load, with the pull-out force at higher displacements now quite well aligned with the other tests using the same bar and temperature. The pre-load and correction may have affected the results of these two tests. If these two tests were excluded from the averaged results, the maximum bond stress for the 8mm deformed bar at 210°C would increase by only 2.2N/mm² and the maximum bond stress of the 12mm deformed bar at 290°C would reduce by only 0.14N/mm². It is considered, therefore, that this has not significantly affected the overall research outcomes.

Figures 3.14 to 3.17 are included to demonstrate the way in which the raw pull-out force and displacement results have been processed to produce the averaged bond stress-slip curves which are presented in the following sections. These results are analysed in sections 3.2.3 to 3.2.5 along with the other test results. The 8mm deformed bar tests at 110°C have been used as an example for the purposes of this explanation. Figure 3.14 shows the raw, unprocessed, results in terms of pull-out force and displacement for the 8mm deformed bar tested at 110°C. The three data sets in figure 3.14 correspond to the three tests which were conducted in this condition.

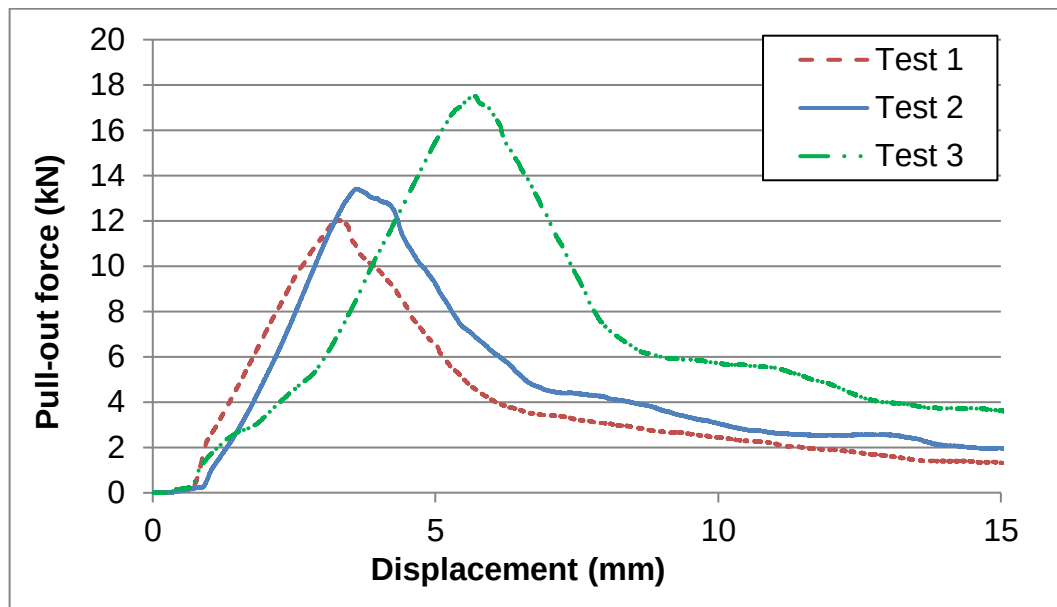


Figure 3.14. Raw, unprocessed, pull-out force and displacement results for 8mm diameter deformed bars tested at 110°C

In order for the final stress-slip curves to show the true average peak bond stress of the three tests, the results were aligned, in terms of displacement, by the point of maximum pull-out force. The aligned pull-out force and displacement curves are shown in figure 3.15. Aligning the curves was necessary in order for the curves to show the actual maximum average bond stress, as this is considered to be an important value in understanding the effect of elevated temperatures on bond strength. Without aligning the curves, the maximum bond stress in the averaged curve would be lower than the mean maximum bond stress of the tests.

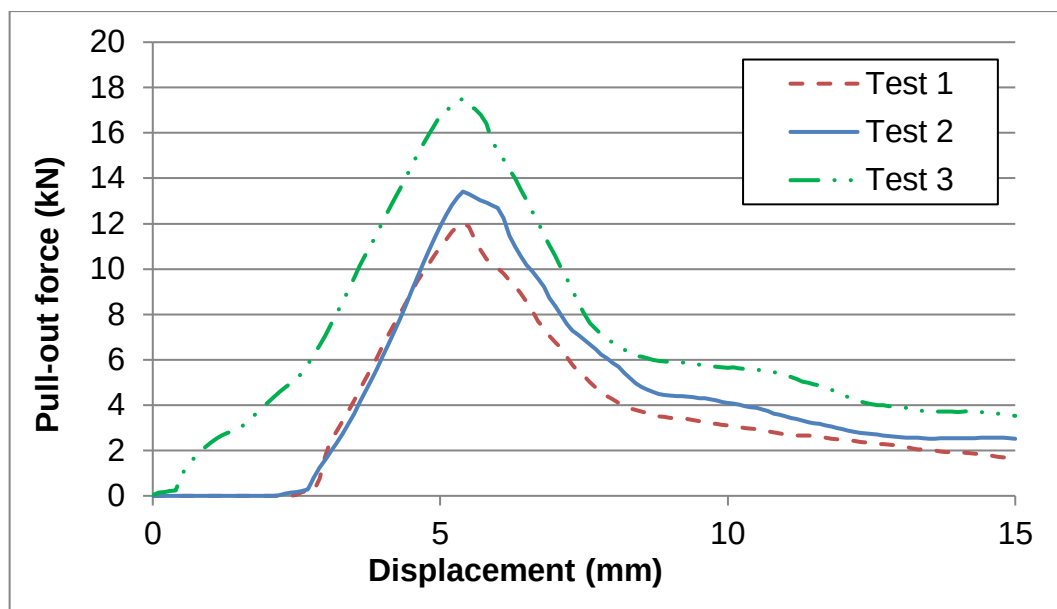


Figure 3.15. Results shown in figure 3.14 after aligning test results by the point of maximum pull-out force of each test

The results of the three tests were averaged by taking the mean value of pull-out force at each displacement based on the aligned results in figure 3.15. The averaged curve for the 8mm deformed bar tested at 110°C is shown in figure 3.16. The initial part of this curve is unrepresentative of the actual bond response in a real structure due to the initial settling in of each specimen and the cradle under load. The extent of the settling in differed for each specimen due to the manual re-positioning of the specimens after securing the grips of the test machine. A further issue with the initial part of the curve is that, in this example, for the first 2mm of displacement, the pull-out force is taken as zero for two of the three tests which were averaged to form this curve. This is a consequence of aligning the results by maximum force, but the issue of settling in was addressed in a subsequent stage of processing, as described later in this section.

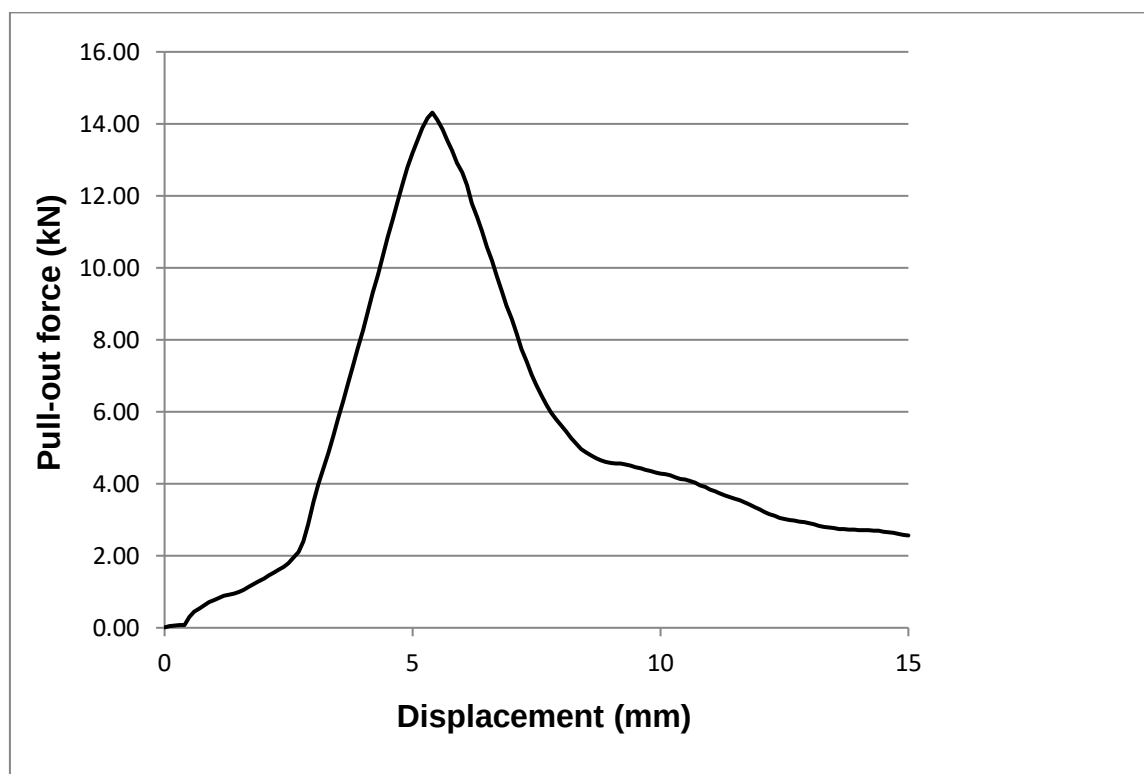


Figure 3.16. Pull-out force and displacement curve derived from averaging the results in figure 3.15

Pull-out force values were converted into bond stress values by dividing the pull-out force by the area of the bond. Slip values were calculated from the measured displacements by subtracting the elastic deformation of the bar. Calculation of slip, s , values was by equation 3.1:

$$s = \delta - l \cdot \frac{F}{A_{bar} \cdot E_{\theta}} \quad (3.1)$$

where δ is displacement measured by the test machine, l is the length of the bar from bond to grip, F is the pull out force, A_{bar} is the cross sectional area of the bar and E_{θ} is the temperature-dependent elastic modulus from BS EN 1994 Part 1-2 (2005b).

An averaged curve of bond stress and slip is shown in figure 3.17. From comparison of figure 3.16 and figure 3.17 the values may appear unchanged except for the units. This similarity of the values of pull-out force and bond stress in their respective units is coincidental in this case and is due to the bonded area of the 8mm bar being 1005mm² and, hence, very close to 1000mm².

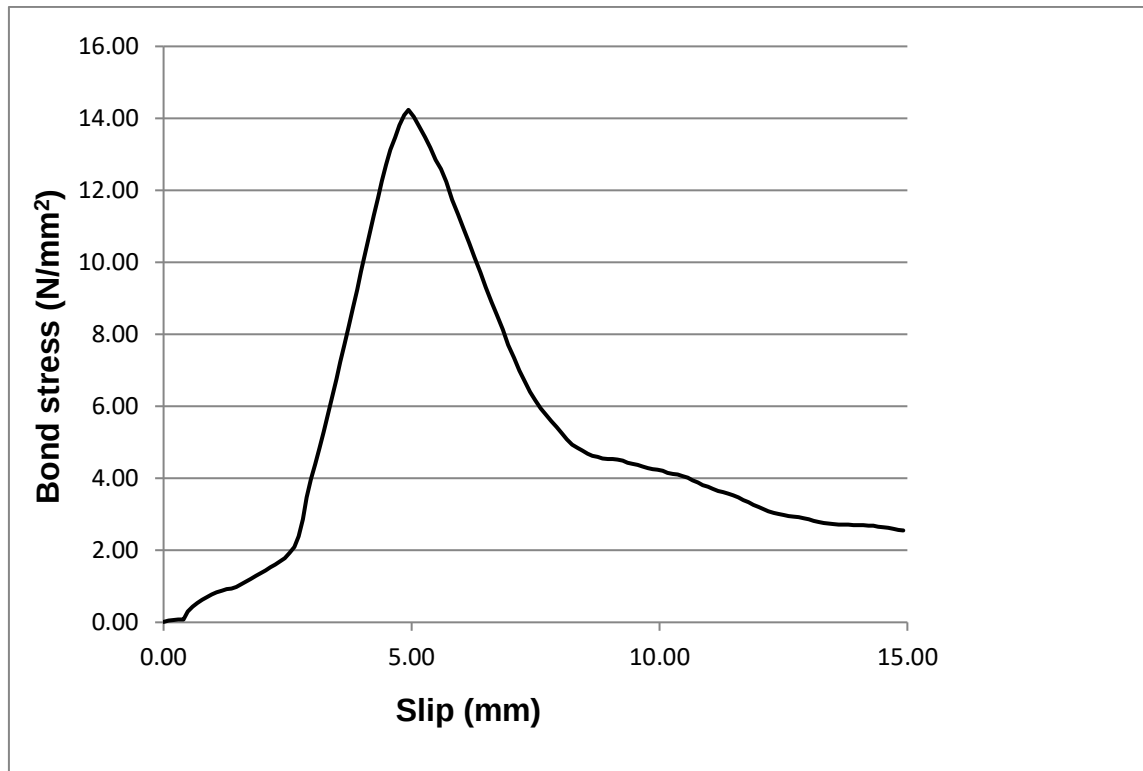


Figure 3.17. Bond stress-slip curve derived from values of pull-out force and displacement from figure 3.16

The curve was processed further in order to address the issue with the initial part of the curve being unrepresentative of an actual bond response due to the effects of the specimen and cradle settling in and the consequence of aligning the results by the point of maximum force. The point of maximum tangent gradient of the curve was identified. The curve was projected back from this point at the maximum gradient to the point of zero bond stress. This gradient was measured over intervals of 0.4mm of bond slip to reduce the influence of any small fluctuations in pull-out force. This resulted in the final bond stress-slip curve as shown in figure 3.18. A disadvantage of this approach is that any fine detail of the initial part of the curve is omitted. It was an approach, however, that could be consistently applied to all results and the resulting shape of the stress-slip curves is generally as expected, based on comparison to

the, semi-empirical, fib local bond model (International Federation for Structural Concrete, 2013) which is explained in section 4.1.

Aligning the curves in a previous step was necessary in order for the curve to show the actual maximum average bond stress and this was considered to be important. The initial settling in could have been reduced if a small pre-load had been applied, but this was avoided due to concerns over damaging the grips during specimen heating, as previously mentioned. The raw, unprocessed, results are included in Appendix E for reference.

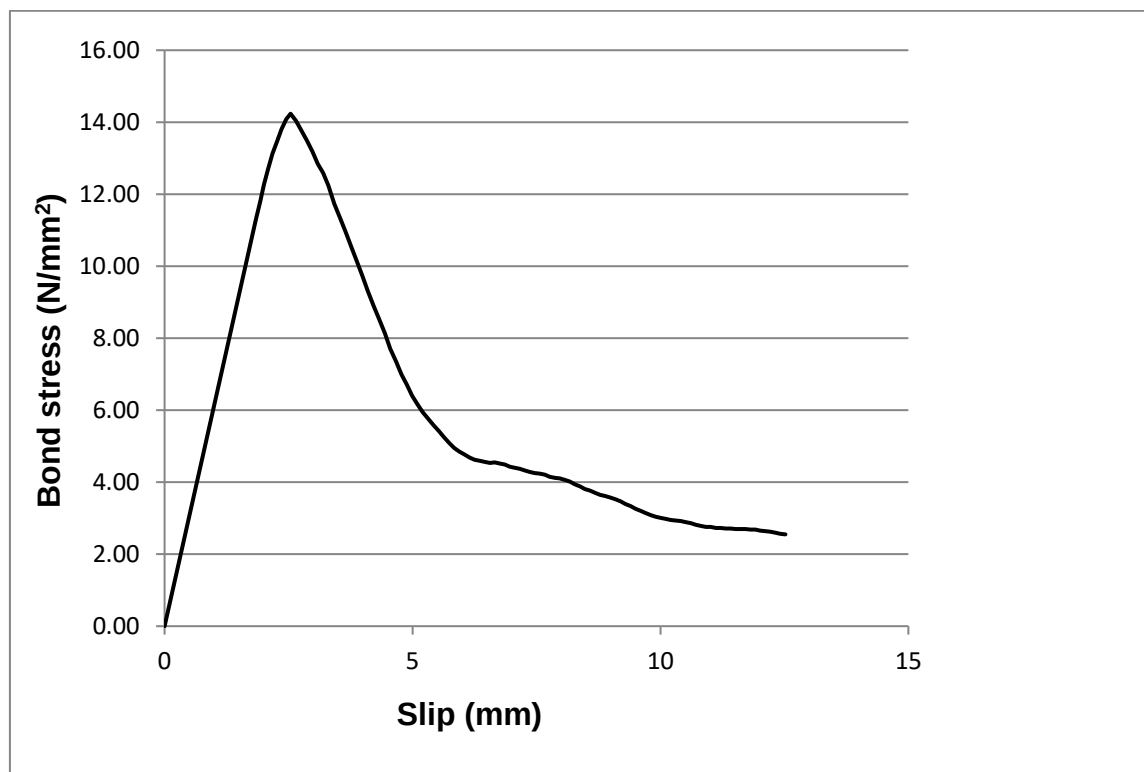


Figure 3.18. bond stress-slip curve for 8mm deformed bar tests at 110°C based on the curve in figure 3.17, but with adjustment for settling in of the specimen and cradle.

3.2.3 Overall results

This section presents the results and analysis of the elevated temperature pull-out tests. The analysis of the pull-out test results largely focuses on three features of the stress-slip curves, as these are the most relevant to the discussion in section 3.3. These features are: the initial gradient of the curve, the magnitude of peak stress, and magnitude of remaining bond stress as it levels off after the peak. The important features of the results presented in section 3.2.3 are explained in section 3.2.4 and 3.2.5. The results are discussed in section 3.3.

Figure 3.19 shows the results of the ambient temperature pull-out tests with the four reinforcing bars. These results are shown in the form of averaged stress-slip curves for each bar type, after the results have been processed as explained in section 3.2.2. Figures 3.20 to 3.23 show the equivalent plots for the tests conducted at each of the elevated temperatures considered.

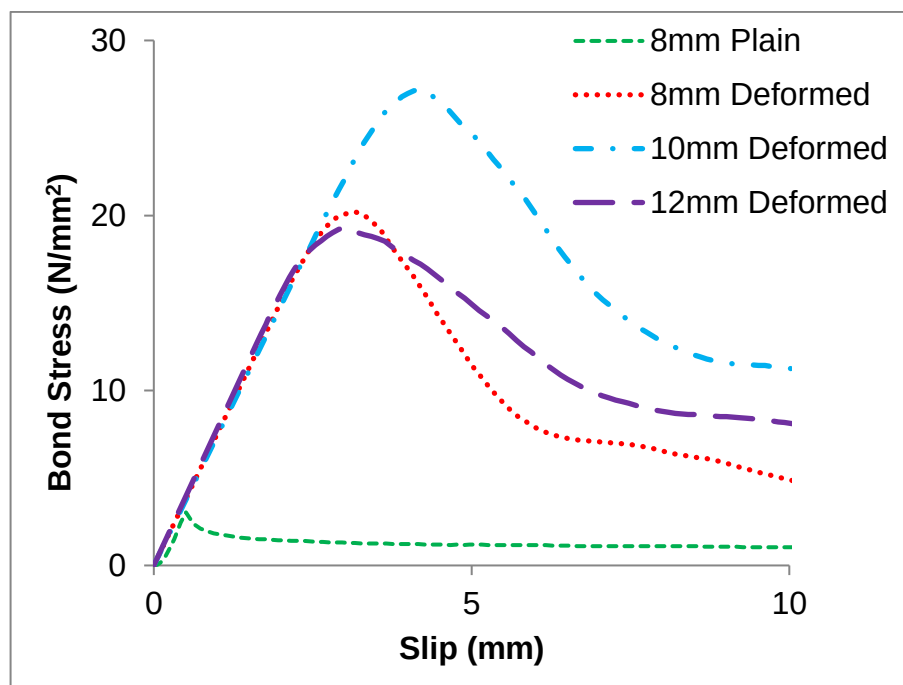


Figure 3.19. Bond stress vs. slip at 15°C for all bar types considered

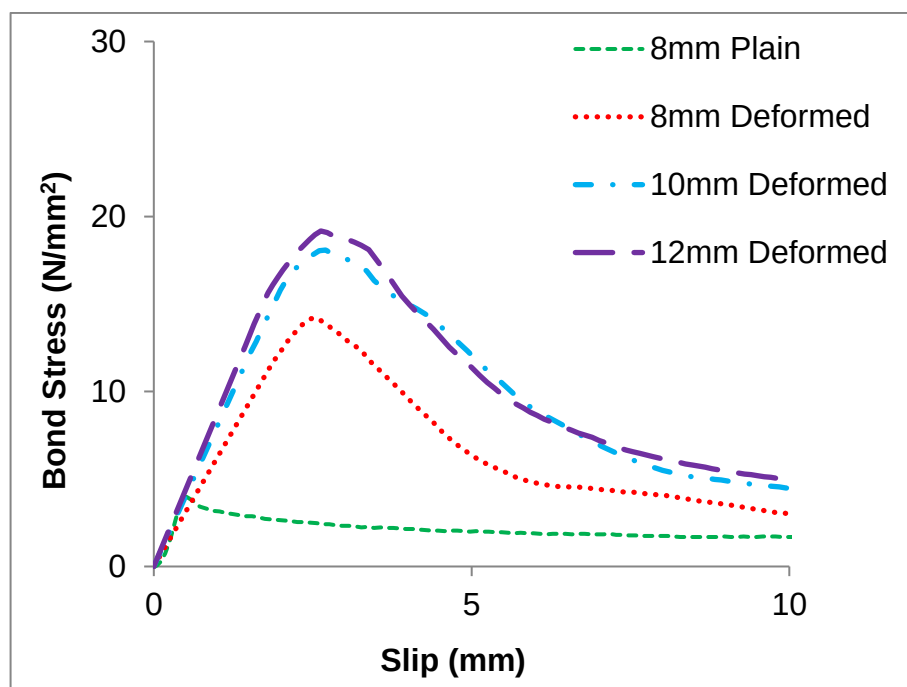


Figure 3.20. Bond stress vs. slip at 110°C for all bar types considered

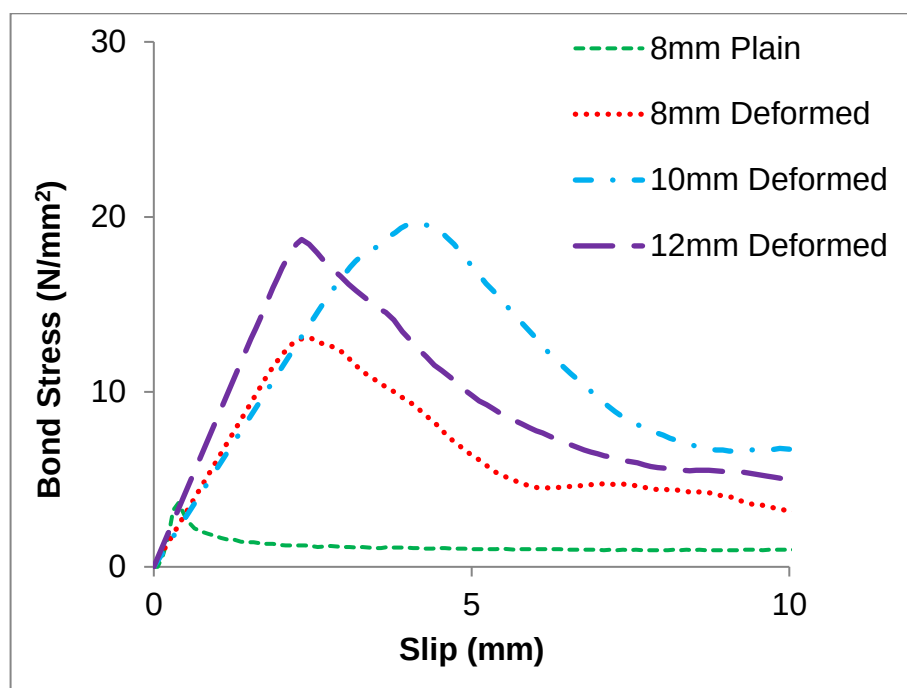


Figure 3.21. Bond stress vs. slip at 210°C for all bar types considered

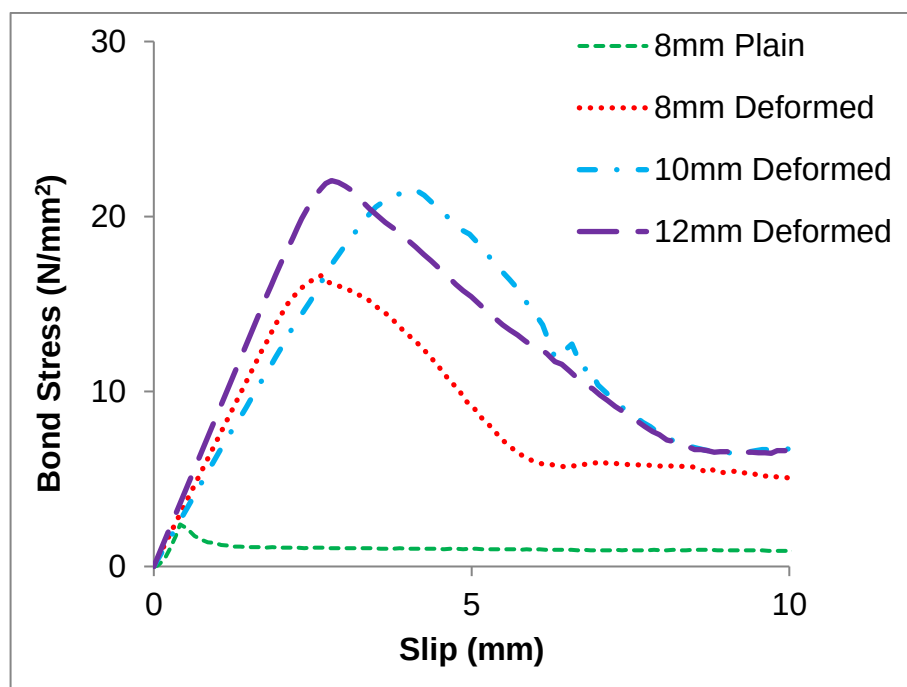


Figure 3.22. Bond stress vs. slip at 290°C for all bar types considered

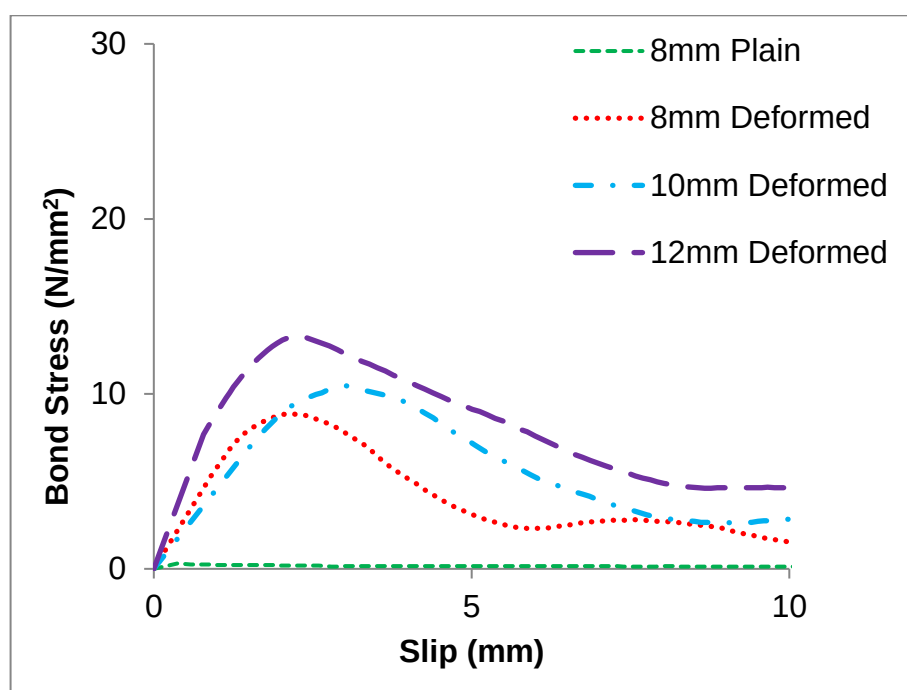


Figure 3.23. Bond stress vs. slip at 570°C for all bar types considered

Figure 3.24 presents bond strength retention, as a proportion of the ambient bond strength, at each temperature considered for different bar diameters. The important features of this figure will be described in sections 3.2.4 and 3.2.5.

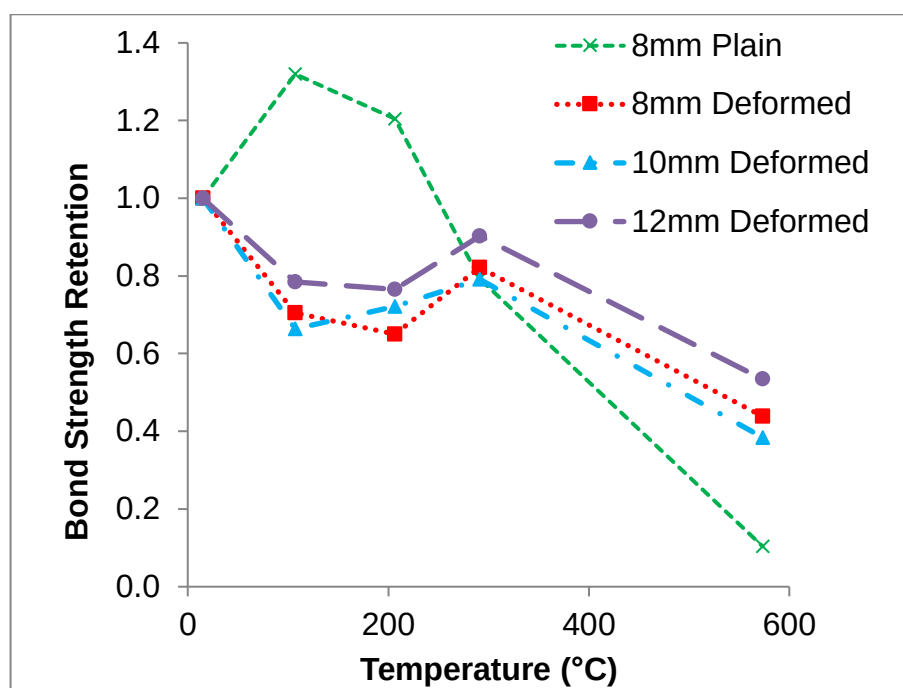


Figure 3.24. Retention of bond strength for the different reinforcement bar types

Figure 3.25 shows the ratio of stiffness at each temperature relative to the stiffness at ambient temperature for each bar diameter. These stiffness values are the maximum tangent stiffness measured over intervals of 0.4mm displacement, to reduce the influence of any small fluctuations in pull-out force. These values are, therefore, gradients from the initial ascending region of the corresponding stress-slip curves.

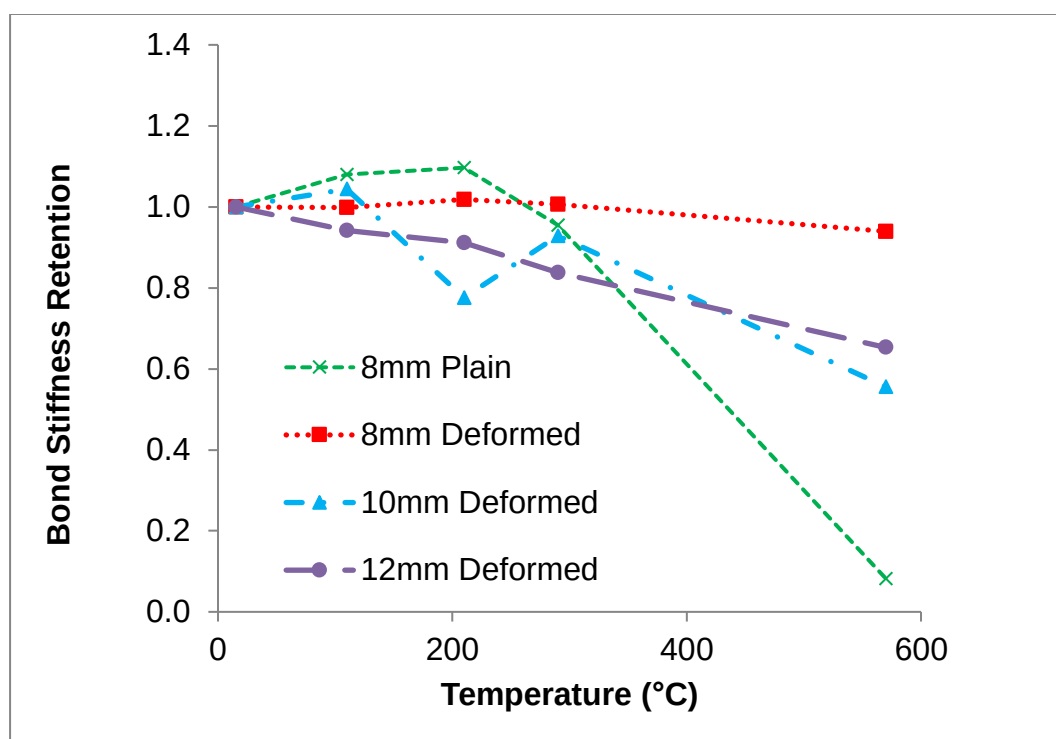


Figure 3.25. Retention of Stiffness

3.2.4 Results and analysis for plain round bars

Figures 3.19 to 3.23 compared the bond response of the different bar types at each temperature. To aid the discussion in section 3.3, it is useful to also compare the responses of each bar at different temperatures. Plain bars are considered in this section, and the response of deformed bars is described in section 3.2.5. Figure 3.26 shows the averaged stress-slip curves for the plain round bars at different temperatures. The results shown in figure 3.26 are the same as those presented for plain bars in figures 3.19 to 3.23, but re-plotted here to aid comparison. The reason for the high bond stress in the 110°C specimen in figure 3.26 is discussed in section 3.3.1.

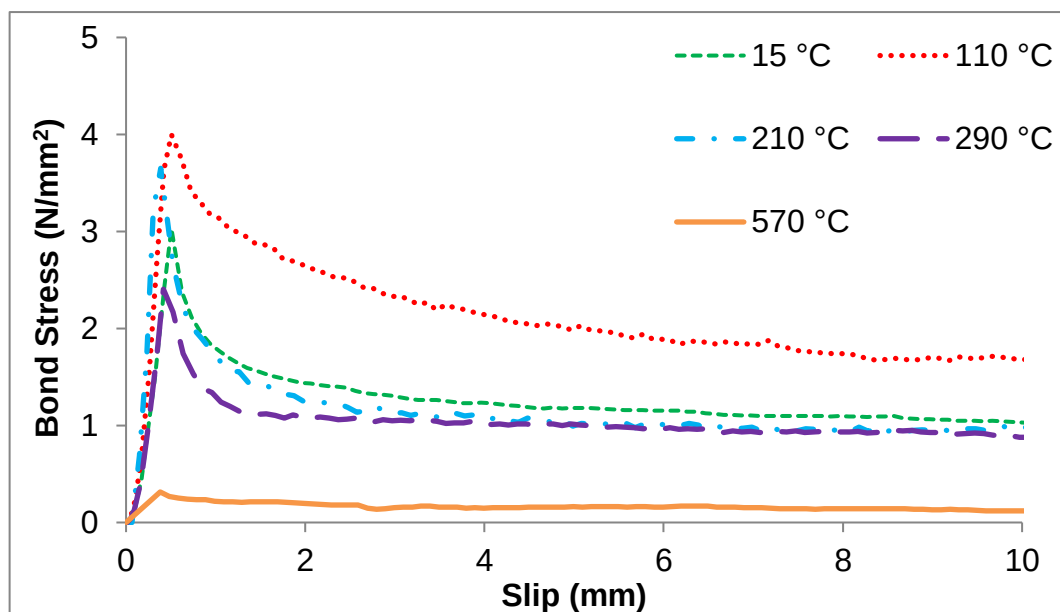


Figure 3.26. 8mm plain round bar bond stress vs. slip at elevated temperature

Peak bond stress was highest at 110°C of the temperatures considered, and was about 30% greater than at 15°C, as shown in figure 3.24. The post-peak bond stress was also higher at 110°C than at other temperatures. Peak bond stress remained greater than ambient strength at 210°C, but decreased with further temperature

increase. At 570°C, the peak bond stress had reduced by almost 90% relative to the ambient condition.

The post-peak bond stress levelled off at a significantly higher level at 110°C than at any other temperature considered, as shown in figure 3.26. At 570°C, the post-peak bond stress levelled off at a far lower level than at any other temperature. At temperatures other than 110°C and 570°C, the level of bond stress after the peak was very similar.

The stiffness of the bond response of plain round bars increased at 110°C and 210°C relative to the ambient temperature condition, as shown in figure 3.25. Bond stiffness of plain round bars was lower at 290°C than at the lower temperatures considered. Bond stiffness of plain round bar was drastically reduced at 570°C relative to any other temperature considered, and had reduced by 92% relative to the ambient temperature condition.

3.2.5 Results and analysis for deformed bars

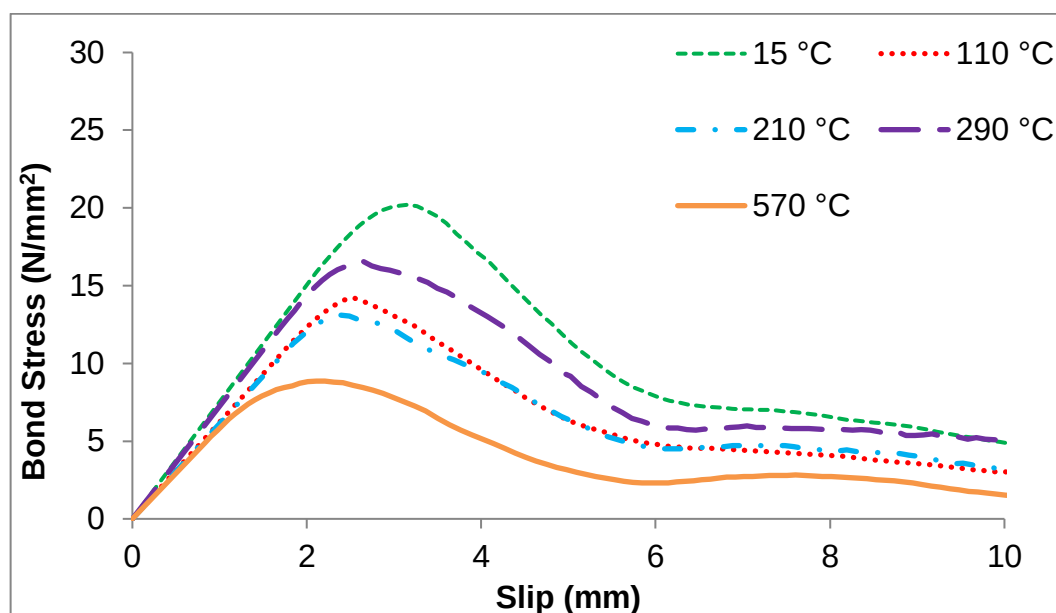


Figure 3.27. 8mm deformed bar bond stress vs. slip at elevated temperatures

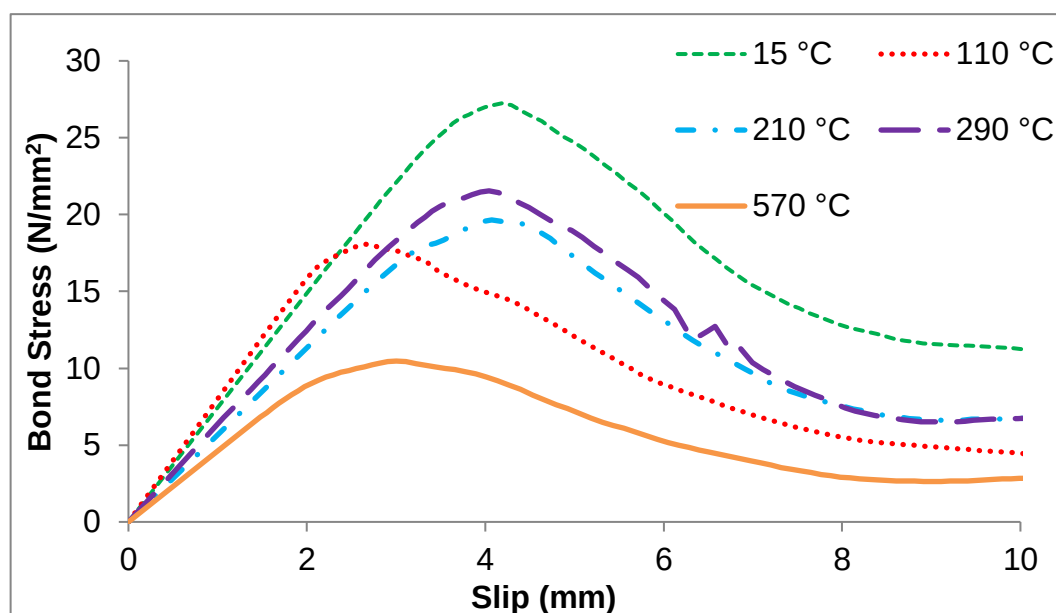


Figure 3.28. 10mm deformed bar bond stress vs. slip at elevated temperatures

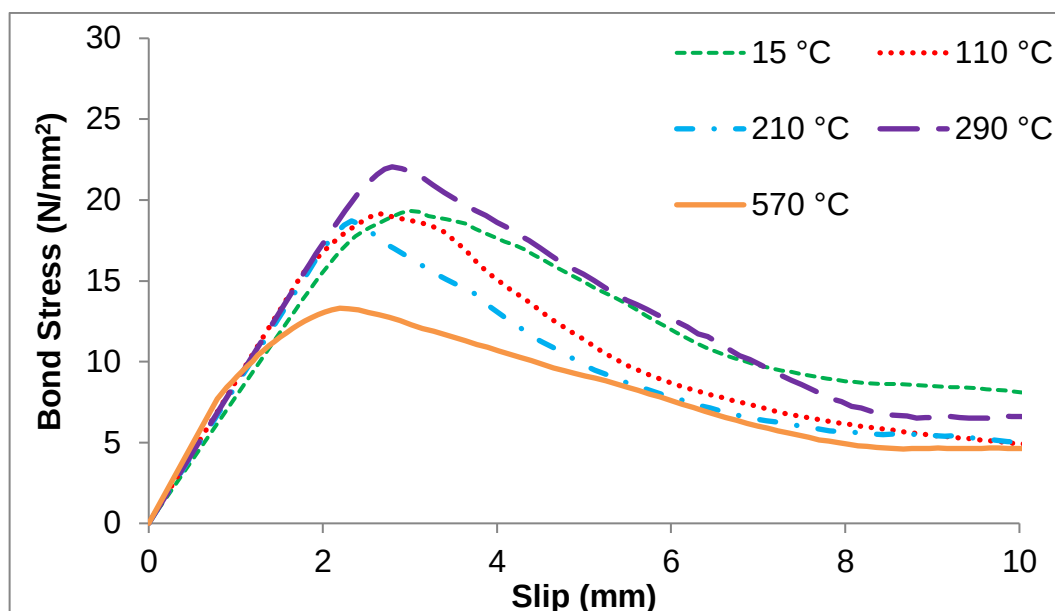


Figure 3.29. 12mm deformed bar bond stress vs. slip at elevated temperatures

From figure 3.24, it is clear that there was a non-monotonic, but generally declining, trend in bond strength retention of deformed bars with increasing temperature. The trend was quite similar for all bar diameters considered. For all bar diameters, there was a reduction in bond strength at 110°C and 210°C relative to the ambient condition. Bond strength was greater at 290°C for all bar diameters than at 110°C and 210°C. At 570°C, the bonds strength had reduced to a level significantly lower than at any other temperature considered.

The bond stress of deformed bars levelled off at a higher level in the ambient condition than at elevated temperatures. At 570°C, the bond stress levelled off at a lower level than at any of the other temperatures considered. The bond stress levelled off from around 6mm of slip for the 8mm deformed bar, and from around 8mm of slip for the 10 and 12mm diameter. In each case, the bond stress levelled off from a slip of around 1mm greater than the rib spacing of the bar.

The stiffness of deformed bars decreased by an average of 28% for all deformed bars at 570°C relative to the ambient condition, as shown in figure 3.25. The variability in the results up to 290°C makes any trend unclear in that temperature range. The 12mm diameter deformed bar retained a higher proportion of its bond stiffness than the smaller diameter bars at all elevated temperatures considered. The 8mm diameter bar retained a higher proportion of stiffness than the 10mm diameter bar at all elevated temperatures except 110°C.

3.3 Discussion relating to elevated temperature bond experiments

3.3.1 Discussion for plain round bars

The ambient temperature stress-slip curve in figure 3.26 follows the expected shape, demonstrating a progression through stages of increasing slip as described in section 2.4. The initial peak in bond stress is primarily due to the strength of the chemical adhesion between the bar and the surrounding concrete. This is the dominant mechanism and determines the strength of the bond. Chemical adhesion is only present at very low values of bond slip. As slip increased, the chemical adhesion was overcome, and the bond stress reduced before levelling off at a lower value. The bond stress in the levelled-off region of the bond stress-slip curve is due to the resistance provided by friction forces. The stages of the bar pull-out for a plain round bar are annotated in figure 3.30. These stages of bond failure are in general agreement with the description given in Fib Bulletin No. 10 (International Federation for Structural Concrete, 2000) for plain bars. By investigating the effect of temperature on the shape of the stress-slip curve, we can infer the effects of elevated temperature on the underlying mechanisms.

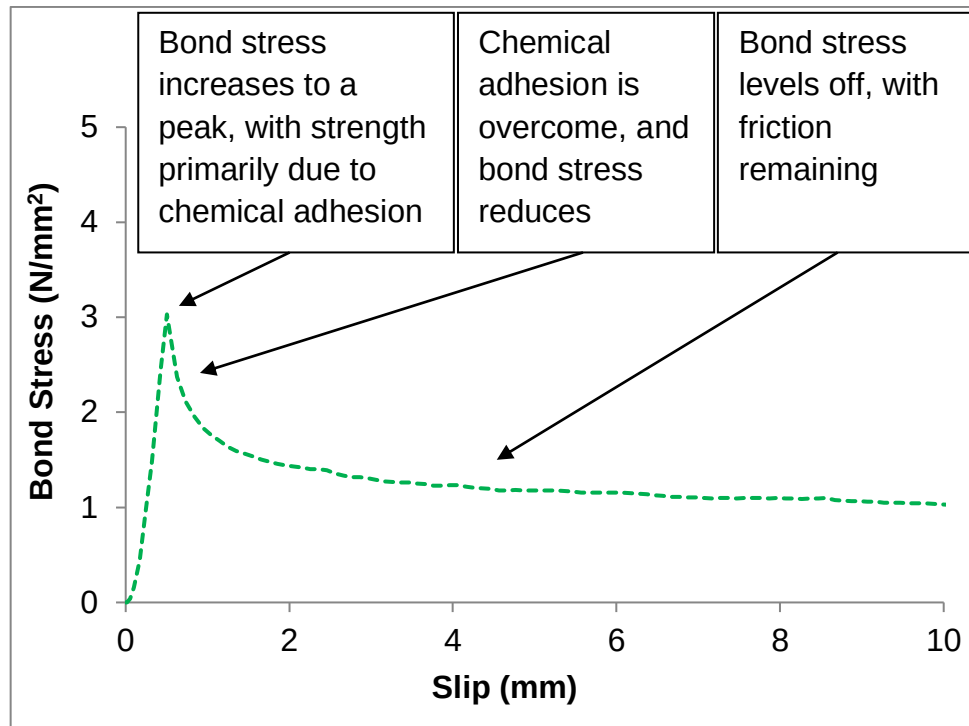


Figure 3.30. Bond stress slip curve for 8mm plain bar at ambient temperature, annotated to describe stages of bar pull-out

The increase in peak bond stress at 110°C and 210°C relative to the ambient condition, as evident from figures 3.24 and 3.26, can be explained by the effects of differential thermal expansion. At these temperatures, the thermal expansion of the bar exceeds that of the concrete, as discussed in section 2.6.4. This resulted in a confining pressure on the bar, increasing friction. This increased resistance from friction acts in superposition with the resistance from chemical adhesion at the point of peak bond stress. The higher bond stress in the post-peak region at 110°C indicates greater friction, and this can also be explained by differential thermal expansion. The stiffness of the bond response is also increased at this temperature, as evident from figure 3.25, and this is also due to the increased confinement on the bar from differential thermal expansion of the steel and concrete.

The bond strength of plain round bars is lower at 290°C than at the lower temperatures considered. Differential thermal expansion will still result in a confining pressure on the bar at this temperature although the cement, which is responsible for the chemical adhesion, will have undergone thermal shrinkage and, potentially, micro cracking. At this temperature, the Portland cement paste will have contracted due to the dehydration of hydration products, such as C-S-H gels and calcium hydroxide (Cruz and Gillen, 1980), and the tensile strength of the cement paste will have been substantially reduced (Tufail et al., 2017). It is reasonable to expect that the resistance provided by chemical adhesion would be reduced as the cement deteriorates in this way. At 290°C, the post-peak bond stress level is similar to the level at 210°C indicating that the level of friction is similar due to the confining pressure on the bar, from differential thermal expansion, still being present. As with bond strength, the stiffness of the bond response is reduced at 290°C, as evident from figure 3.25, due to the deterioration of the cement.

The large reduction in peak bond stress at 570°C relative to other temperatures, as evident from figures 3.24 and 3.26, is due to the reduced chemical adhesion and the effects of differential thermal expansion. Chemical adhesion will be reduced at this temperature due to the deterioration of the cement paste, including the breakdown of portlandite in the cement from around 400°C (Hager *et al.*, 2015). At this temperature, differential thermal expansion will also have contributed to the reduction in peak bond stress. In this case, there is a partial separation of the bar and concrete due to the concrete expanding by more than the steel at this temperature. This was explained in section 2.6.5. The greatly reduced bond stress in the post-peak region at 570°C indicates reduced friction. In this case, friction will have been reduced by differential thermal expansion and partial separation of the

bar and concrete. The stiffness of the bond response of the plain bar is also greatly reduced at 570°C, as evident from figure 3.25. This reduction in stiffness is also due to the reduction in chemical adhesion and the partial separation of the bar and concrete due to differential thermal expansion.

3.3.2 Discussion for deformed bars

The ambient temperature bond stress-slip curves, shown in figure 3.19, for deformed bars follow the expected shape, demonstrating the progression through the stages of bond behaviour with increasing slip as described in section 2.4. There is an initial peak in bond stress due to the resistance of mechanical interlock, and a subsequent reduction in bond stress as the mechanical interlock is overcome with increasing slip. The bond stress levels off at a reduced level, where resistance is largely provided by friction forces. The stages of bar pull-out for a deformed bar are described in figure 3.31. These stages of bond failure are in general agreement with the description given in Fib Bulletin No. 10 (International Federation for Structural Concrete, 2000) for deformed bars. As for the plain bars, the effects of elevated temperature on the underlying mechanisms can be deduced by considering the effect on the shape of the stress-strain curves. The shape of the stress-strain curve up to the peak stress resembles the compressive stress-strain response because the resistance is provided by the concrete as it is crushed below the ribs of the bar.

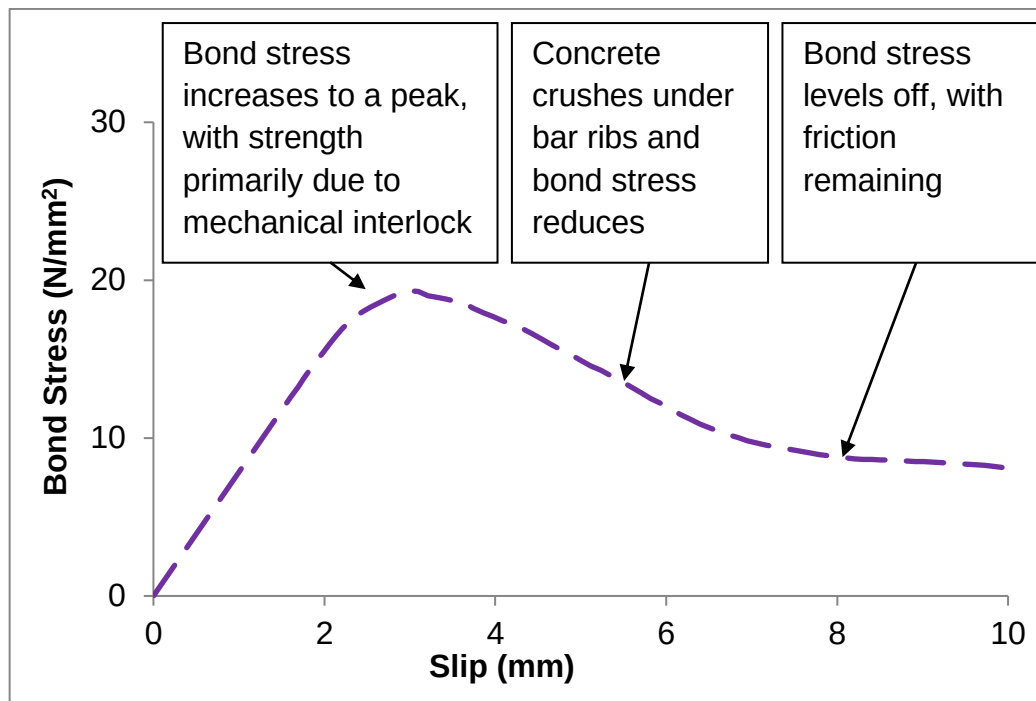


Figure 3.31. Bond stress slip curve for 12mm deformed bar at ambient temperature, annotated to describe stages of bar pull-out

Figure 3.24 shows a down-up-down trend in bond strength with increasing temperature for deformed bars. The trend reflects the trend which has been observed for concrete compressive strength with increasing temperature. This is because the bond strength of deformed bars is primarily due to mechanical interlock, and mechanical interlock depends on concrete strength being sufficient to resist bearing stresses of the ribs. The trend in compressive strength was described in section 2.6.1. Although the pattern is similar, bond strength declines more rapidly than compressive strength with increasing temperature. This is due to differential thermal expansion between the steel and concrete generating further stress at the bond interface and contributing to micro cracking of the concrete.

The proportion of bond strength retained at elevated temperatures was greater for the 12mm deformed bar. Although the 12mm diameter bar performed superiorly, the 10mm diameter bar performed inferiorly to the 8mm diameter bar in terms of bond

strength retention at elevated temperature. Therefore, there doesn't appear to be a simple trend between bond strength retention at elevated temperature and increasing bar diameter. This agrees with the findings of Hertz (1982) that the 12mm bar outperformed both smaller and larger bar diameters.

The general reduction in bond stiffness with increasing temperature resembles the findings of Weighler and Fischer (1972) for concrete compressive stiffness at elevated temperature, as explained in section 2.6. The stiffness of deformed bar bond will be related to the stiffness of concrete given that the resistance to slip of deformed bars is provided by the concrete under the ribs. Random aggregate positioning relative to the ribs may be responsible for some scattering of stiffness values.

3.3.3 Comparison of plain round and deformed bar

Plain bars retained a greater proportion of their ambient bond strength than deformed bars at elevated temperatures under 290°C. At 290°C, the plain and deformed bars retained a similar proportion of their respective ambient bond strengths. The proportion of bond strength retained at 570°C was greater for deformed bars than plain bars. The difference in observed behaviour of the plain and deformed bars can be attributed to the different mechanisms which govern the bond of each bar type. Each of these different mechanisms is affected differently by elevated temperatures, as is discussed in sections 3.3.1 to 3.3.3.

3.3.3.1 Comparison of bar types at temperatures under 290°C

At elevated temperatures under 290°C, plain bars have increased bond strength, as shown in figure 3.24, due to beneficial effects from differential thermal expansion. This effect will be less significant for deformed bars relative to the role of mechanical

interlock which depends more on concrete strength. At these temperatures, the reduction in concrete compressive strength reduced the bond performance of deformed bars. This will be the most significant effect of these temperatures on the bond of deformed bars, but it will be relatively less important for the bond of plain bars, as they can fail without crushing of the surrounding concrete.

At elevated temperatures under 290°C, the stiffness of the bond response of the plain round bars increased relative to the ambient condition, as evident from figure 3.25, due to the confining pressure from differential thermal expansion, as explained in section 3.3.1. At the same temperatures, the proportion of ambient bond stiffness retained was lower for deformed bars than it was for plain round bars because the presence of a confining pressure due to differential thermal expansion is less significant for deformed bars which are more influenced by changes in concrete compressive stiffness which will not have changed appreciably at these temperatures, as described in section 2.6. The level of scatter makes the trend for deformed bars unclear at these temperatures. The change in bond stiffness at these temperatures relative to the ambient condition was, on average, less than 10% for the two bar types and, as at ambient temperature, their bond stiffness was relatively similar in this temperature range.

3.3.3.2 Comparison of bar types at 290°C

At 290°C the bond retention of deformed bars and plain bars was similar, and both had deteriorated modestly relative to the ambient condition, as evident from figure 3.24. At 290°C, relative to 110°C and 210°C, bond strength of deformed bars was increased, but the bond strength of plain round bars was reduced. This relative increase in bond strength of deformed bars was due to concrete compressive

strength being higher at 290°C than 110°C and 210°C, improving the resistance of mechanical interlock. Although concrete compressive strength is higher at 290°C, the chemical adhesion will be reduced, as discussed in section 3.3.1. The reduction in chemical adhesion is more significant for plain round bars than deformed bars as it is the primary contributor to bond strength of plain round bars. Chemical adhesion is of less significance, in relative terms, to the bond of deformed bars than plain round bars because the bond strength is primarily due to the larger resistance provided by mechanical interlock of the bar ribs (Lutz and Gergely, 1967).

At 290°C, the stiffness of the bond response of both plain round bars and deformed bars, on average, is slightly lower than at ambient temperature, as evident from figure 3.25. The bond stiffness of deformed bars has reduced only very modestly at this temperature because the compressive stiffness of the concrete, which resists the bearing pressure from the ribs, is still largely retained, as described in section 2.6.

3.3.3.3 Comparison of bar types at temperatures over 290°C

Bond strength retention of plain round bars had reduced drastically at 570°C and to a much greater extent than for deformed bars. The reason for this is that the mechanisms which dominate plain bar bond have been severely diminished at 570°C. Chemical adhesion has weakened by the breakdown of the cement paste through thermo-chemical reactions. The loss of chemical adhesion between cement and steel is more significant for plain bars than deformed bars, although the weakening of the cement is also a factor in the reduction of concrete compressive strength which is relied upon for mechanical interlock of deformed bars.

Differential thermal expansion will be much more pronounced at 570°C and will cause separation of the bar surfaces from the surrounding concrete. These effects will be much more severe for plain bars than for deformed bars as the ribs of deformed bars will still provide mechanical interlock while surface separation will severely diminish any remaining chemical adhesion or friction.

The stiffness of the bond response of plain round bars is reduced by over 90% at 570°C compared to an average of just 28% reduction for the deformed bars, as evident from figure 3.25. The compressive stiffness of concrete will be considerably lower at this temperature and this is reflected in the reduction in stiffness of the bond response of deformed bars. The effects of differential thermal expansion and the loss of chemical adhesion on the bond stiffness of plain bars are, however, much more severe. Of the temperatures considered, it was only at 570°C that there was much difference in the bond stiffness of plain round bars and deformed bars, with plain round bars having much lower bond stiffness in this condition.

3.3.4 Comparison with previous elevated temperature bond studies

The relationship between bond strength and temperature for deformed bars as identified in this research is compared with the findings of previous studies in figure 3.32. The results included in figure 3.32 were those with the test input parameters which were most comparable to the current experiments considering factors such as bar diameter and test condition. Some of these studies considered temperatures above 600°C, but the results included in figure 3.32 are only those up to this temperature as this is the limit of the current study. The results from the full temperature range of each study are shown in section 2.8.1.

The test results by Morley and Royles (1983) shown in figure 3.30 used aggregate which was described as natural gravel and specimens were tested when hot. Test results by Hertz (1982) shown in figures 3.30 and 3.31 used a mixture of quartz, granite and limestone aggregate and specimens were tested after cooling. The results by Diederichs and Schneider (1981) shown in figures 3.30 and 3.31 used siliceous aggregates and specimens were tested when hot. Test results by Sager *et al.* (1980) included in figure 3.30 used siliceous aggregates and were tested hot.

The results of these studies share some similar features but also have significant differences. There is a broad consensus between these studies that bond strength of deformed bars is quite well retained at around 300°C where bond strength is at least around 79% of ambient strength. All of these results also show a general decline in bond strength with increasing temperature, although the exact trend is not consistent between studies. There is also a general agreement between the studies that bond strength is lower at 600°C than at any lower temperature considered. The trend in the current study is in particularly good agreement with the results of Morley and Royles (1983).

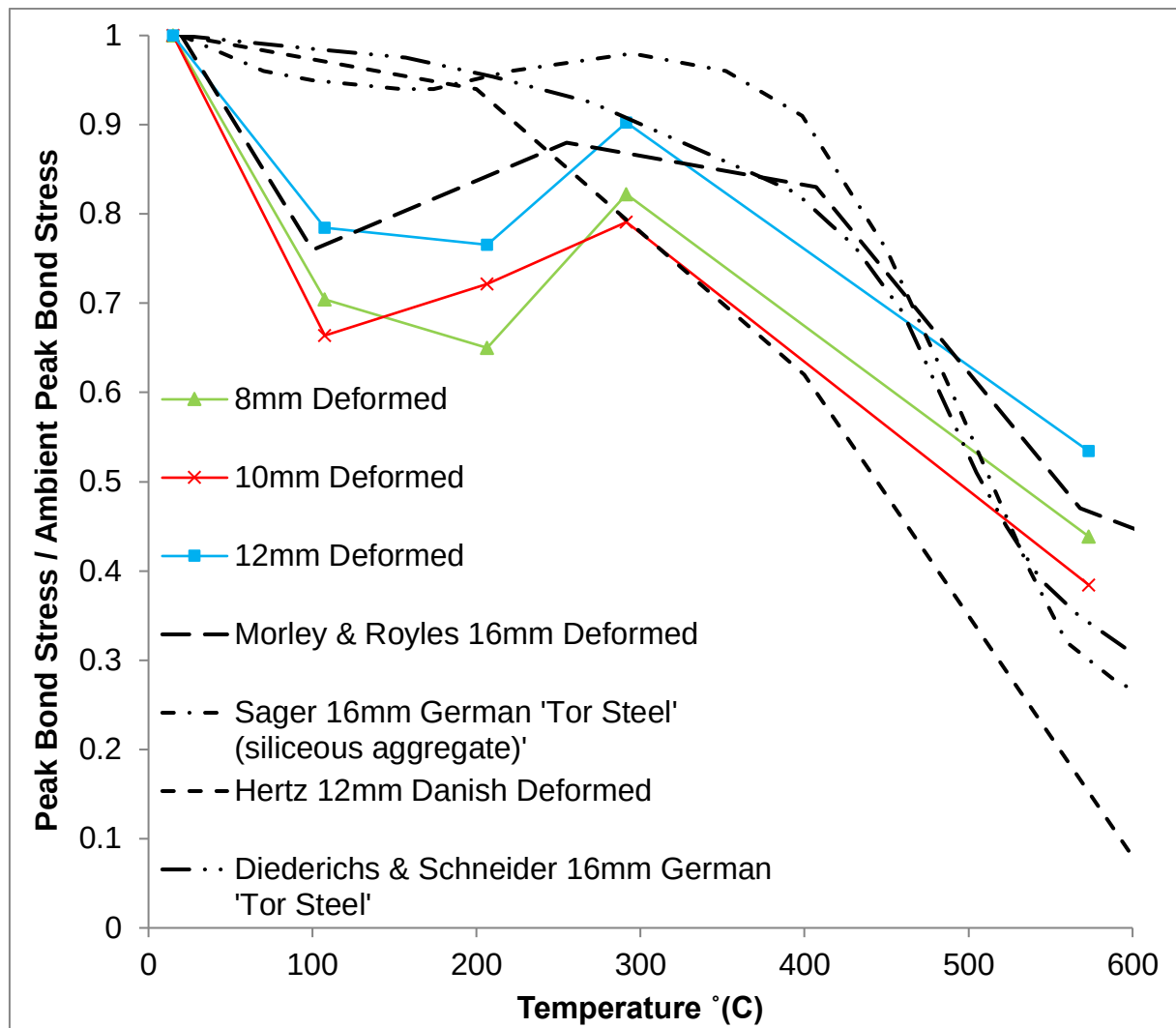


Figure 3.32. Comparison of bond strength retention at elevated temperature for deformed bars from different studies as a proportion of ambient strength (Diederichs and Schneider, 1981; Hertz, 1982; Morley and Royles, 1983; Sager *et al.*, 1980)

One feature of the current results which is dissimilar to the previous studies, with the exception of Morley and Royles (1983), is the presence of the down-up-down trend which was discussed previously. As this trend is a reflection of the trend in compressive strength with temperature, differences in concrete mix and aggregate type between studies are one explanation of this difference between studies. A further explanation is that the trend may not have been identified in some studies due to a sparsity of data at temperatures around 100 to 200°C. Diederichs and

Schneider (1981) presented these results as a smooth curve without individual points, but stress-slip curves results were presented at 20°C and 300°C and not at temperatures between these values so the 100°C to 200°C range does not appear to have been investigated. Hertz (1982) did not mark the individual data points, but the lines in the plot appear to connect points at approximately 20°C, 200°C and 400°C and higher temperatures, although there may have been intermediate points on these lines.

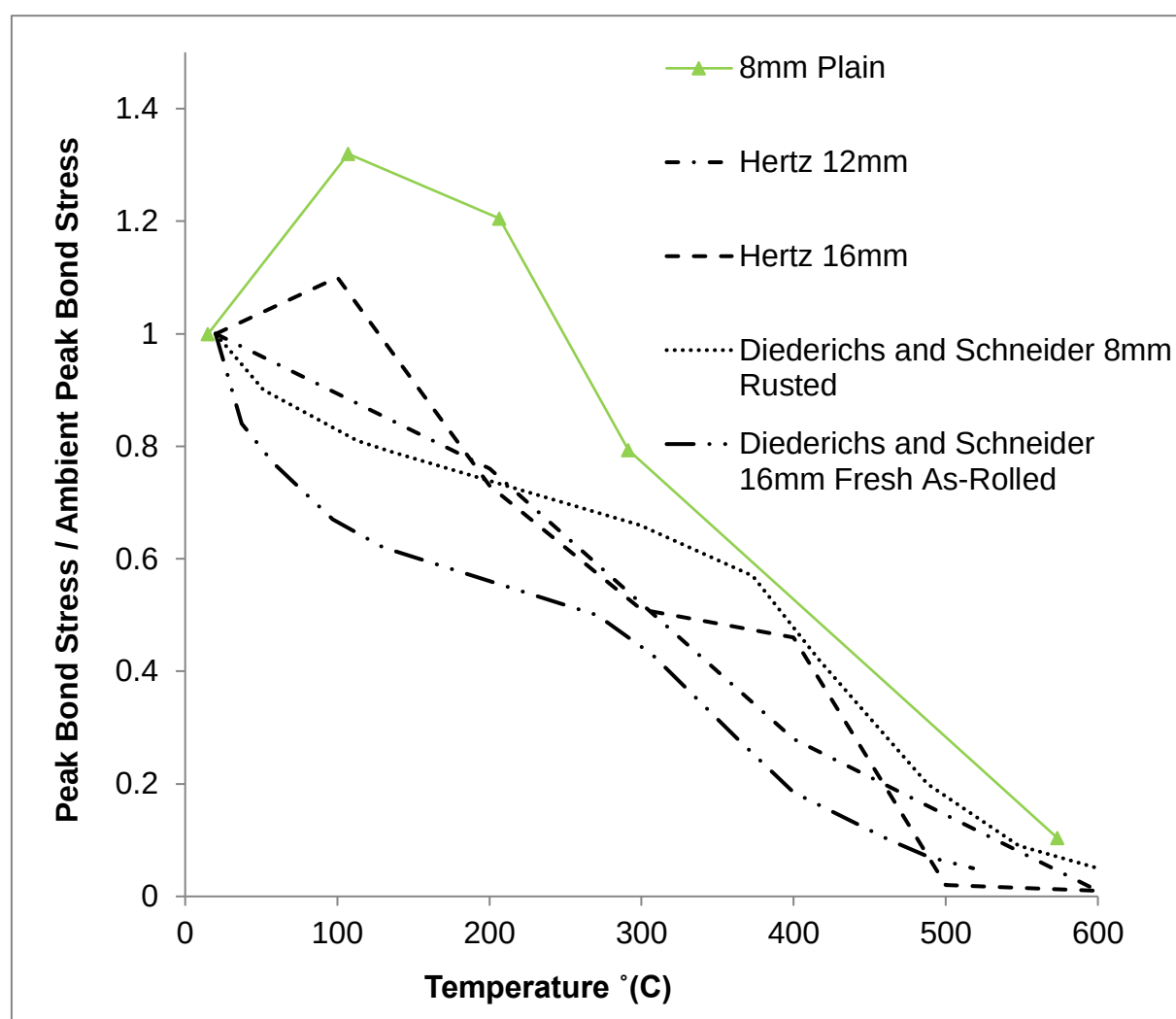


Figure 3.33. Comparison of bond strength retention at elevated temperature for plain bars from different studies as a proportion of ambient strength (Diederichs and Schneider, 1981; Hertz, 1982)

Figure 3.33 shows the relationship between bond strength and temperature for plain bars as identified in this research along with the results of previous studies for comparison. The results which have been included from previous studies are those that were considered the most comparable in terms of factors such as bar diameter and test conditions. As for deformed bars, the results have only been compared up to 600°C as this is the limit of the current study.

The results all demonstrate a general decrease in bond strength of plain bars with increasing temperature. There is a consensus between studies that bond strength is drastically reduced for plain bars at around 600°C, where it is consistently 10% or less of the ambient temperature strength.

As the results by Hertz (1982) are for tests conducted after cooling, there is no reason to expect an increase in bond strength in the temperature range of 100°C to 200°C in those results, given that this increase is due to differential thermal expansion. Hertz results for the 16mm diameter bar did show a modest increase in bond strength in this temperature range, but there is some scatter in the results and this increase may be within the margin of error, due to factors such as aggregate positioning.

The results for the tests by Diederichs and Schneider (1981) presented in figure 3.33 are taken from figure 2.39. As explained in section 2.8.1.3, the results in this figure contradict those of the stress-slip results for the same study which are shown in figure 2.40. The stress-slip curves, shown in figure 2.40, do show an increase in bond strength at 80°C and 200°C relative to 20°C. The increase in bond strength at these temperatures would be expected given that the specimens were tested hot

and this is similar to the observation for 8mm plain round bars in the current research.

The stress-slip curves from Diederichs and Schneider (1981), shown in figure 2.40 have a different shape from those in the current study, as shown in figure 3.26. In the stress-slip curves in figure 2.40 the bond stress remains relatively flat with increasing bond slip after the initial rise in bond stress. This contrasts with the stress-slip curves in figure 3.26, which show a much sharper decline in bond stress with increasing bond slip after the initial peak. The plain round bars which were used by Diederichs and Schneider (1981) were described as heavily rusted, but the plain round bars used in this study were only lightly rusted. As rust will increase surface roughness, this explains why the amount of friction would be higher. The increased friction would have produced this different shape of stress-slip curve, with higher bond stress remaining after the chemical adhesion has been overcome.

4 Proposed local bond stress-slip model

This section of the thesis describes a proposed local bond stress-slip model which is based on the fib local bond stress-slip model (International Federation for Structural Concrete, 2013). The fib model has been modified to account for elevated temperatures using the experimental results from this study. The modified model will be useful for subsequent research investigations and design. Features of this modified model were used in the numerical modelling part of this research, as described in later chapters.

4.1 The current fib bond model

The fib bond model discussed here is the local bond stress-slip model described in section 6.1.1.1 of the fib Model Code for Concrete Structures (International Federation for Structural Concrete, 2013). This is a semi-empirical model of the bond stress-slip relationship at normal temperatures. The fib bond model is represented by Figure 4.1.

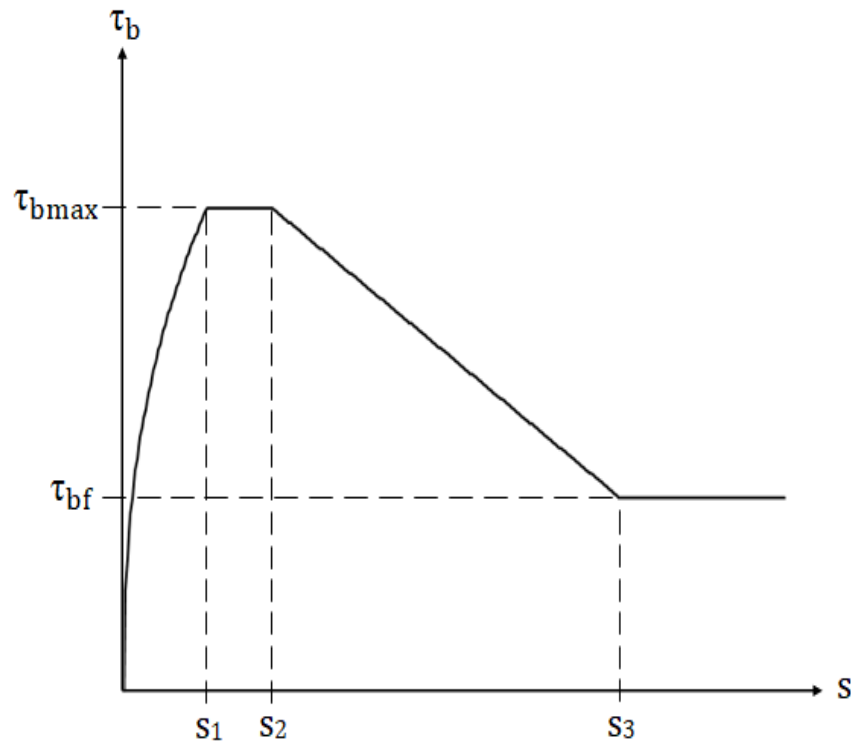


Figure 4.1. Bond stress-slip relationship to fib model

The fib Model Code (International Federation for Structural Concrete, 2013) recommends that the effect of elevated temperatures on the bond of deformed bar bond strength may be accounted for by applying a reduction similar to the reduction in concrete tensile strength. The reduction factor for characteristic tensile strength of concrete at elevated temperatures from Eurocode 2 Part 1-2 (BSI, 2019) is shown in figure 2.25. Considering the results of this research, along with previous studies (Morley, 1982; Morley and Royles, 1983; Royles and Morley, 1983), as shown in figure 3.32, this appears to be a very conservative approach.

The bond model is characterised by equations (4.1) to (4.5). Equation (4.1) represents the initial parabolic phase where bond stress is increasing with increasing bond slip until bond stress reaches its maximum value. Equation (4.2) represents a plateau at maximum bond stress with some further increase in bond slip. A

subsequent decline in bond stress is represented by Equation (4.3), where bond stress falls from its peak value until reaching the point where only friction remains. Equation (4.4) represents a flat bond stress due to friction as bond slip continues to increase. The equations are as follows:

$$\tau_b = \tau_{b\max} \left(\frac{s}{s_1} \right)^\alpha \text{ for } 0 \leq s \leq s_1 \quad (4.1)$$

$$\tau_b = \tau_{b\max} \text{ for } s_1 \leq s \leq s_2 \quad (4.2)$$

$$\tau_b = \tau_{b\max} - (\tau_{b\max} - \tau_{bf}) \frac{(s-s_2)}{(s_3-s_2)} \quad (4.3)$$

$$\text{for } s_2 \leq s \leq s_3 \quad (4.4)$$

$$\tau_b = \tau_{bf} \text{ for } s_3 \leq s \quad (4.5)$$

where the term τ_b is bond stress and the term α is a coefficient defining the parabolic curve with the value 0.4. The parameters s_1 and s_2 are slip values of 1mm and 2mm respectively for 'good bond' conditions. The parameter s_3 is the clear distance between ribs. The fib bond model suggests that maximum bond stress, $\tau_{b\max}$, and bond stress due to friction, τ_{bf} , can be calculated from equations (4.6) and (4.7) in the case of 'good bond' conditions. Criteria to define 'good bond' conditions can be found in section 6.1.3.2 of fib Model Code for Concrete Structures (International Federation for Structural Concrete, 2013).

$$\tau_{b\max} = 2.5 \sqrt{f_{cm}} \quad (4.6)$$

$$\tau_{bf} = 0.4 \tau_{b\max} \quad (4.7)$$

Further guidance on the fib bond model can be found in the fib Model Code (International Federation for Structural Concrete, 2013).

4.2 Proposed modified model for elevated temperature bond

A temperature dependent local bond model has been developed. The process through which the model was developed is summarised in figure 4.2.

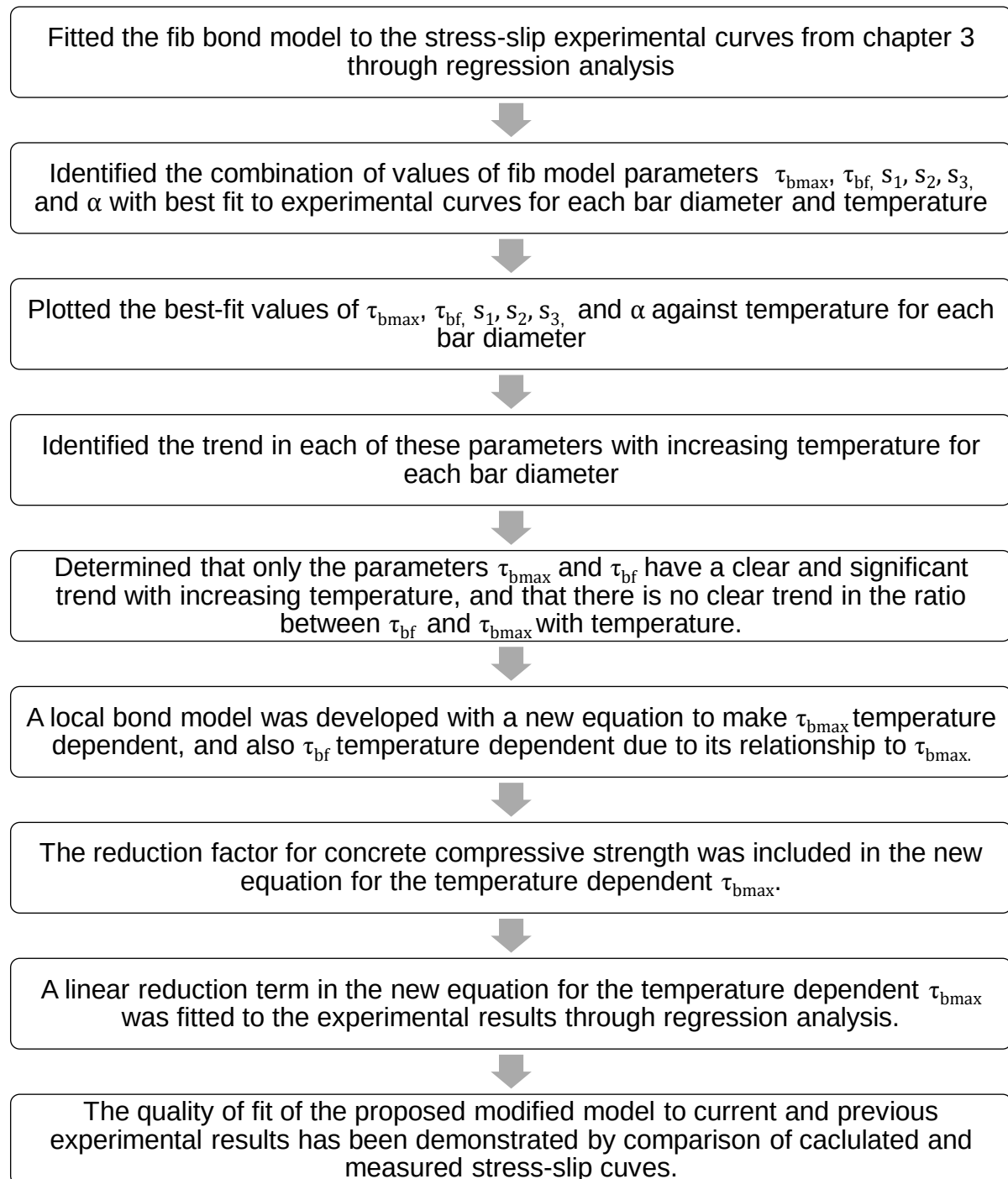


Figure 4.2. Development process of the proposed modified local bond model

It is evident from figures 2.24 and 3.24 that the retention of bond strength at elevated temperature for deformed bars follows a similar trend to concrete compressive strength with increasing temperature. It is also evident that the decline in bond strength is less rapid than the decline in concrete tensile strength. This section describes a new model which had been developed in the current research by modifying the fib bond model. In this new model, the bond strength is based on the reduction in concrete compressive strength at elevated temperature rather than the reduction in concrete tensile strength.

Hence, the proposed local bond stress-slip model treats bond strength at elevated temperature as a function of concrete compressive strength at elevated temperature rather than tensile strength as has been previously suggested (International Federation for Structural Concrete, 2013). A reduction factor, $k_{\text{bond},\theta}$, has been included in the proposed modified model for bond strength retention at elevated temperature and it is defined by Equation (4.8).

$$k_{\text{bond},\theta} = \left[1 - \frac{(\theta-20)}{1500} \right] k_{c,\theta} \quad \text{for} \quad \phi = 8, 10 \text{ or } 12 \text{ mm} \quad (4.8)$$

which can be applied as follows:

$$\tau_{\text{bmax},\theta} = k_{\text{bond},\theta} \cdot \tau_{\text{bmax}} \quad (4.9)$$

where,

θ is the temperature at the steel to concrete interface in °C within the range 20°C to 570°C

f_{cm} is the mean value of compressive strength of concrete

$\tau_{\text{bmax},\theta}$ is the maximum bond stress at θ °C

$k_{c,\theta}$ is the reduction factor for the compressive strength of concrete at elevated temperature which must be sourced from test data

$\emptyset$ is the diameter of the reinforcing bar

$\tau_{b,\theta}$ is the bond stress at slip s and at $\theta^\circ\text{C}$

$\tau_{bf,\theta}$ is the parameter τ_{bf} but calculated from $\tau_{b\max,\theta}$ rather than $\tau_{b\max}$

$1 - \frac{(\theta-20)}{1500}$ is a linear reduction factor derived from regression analysis, as explained in section 4.3.

These modified parameters should be substituted into equations (4.1) to (4.4) to modify the model for elevated temperatures.

4.3 Derivation of the modified model

The modified model was derived from the bond stress-slip curves which are presented in section 3.2. Regression analysis was used to find values for the parameters α , s_1 , s_2 , s_3 , and the ratio between τ_{bf} and τ_{bmax} which would give a curve, using equations (4.1) to (4.4), with the best-fit to the stress-slip curves from the experimental data. The best-fit value of τ_{bmax} was taken directly from the experimental results.

A range of different values for the parameters α , s_1 , s_2 , s_3 , and the ratio between τ_{bf} and τ_{bmax} were used in the equations (4.1) to (4.4) to produce different sets of stress-slip curves. This was done using a data table in a spreadsheet and it included different combinations of the different parameter values. By comparing each of these sets of stress-slip curves with the stress-slip curves from the pull-out experiments in chapter 3, the R^2 value was calculated for each combination of values of α , s_1 , s_2 , s_3 , and the ratio between τ_{bf} and τ_{bmax} . The parameter values in the combination which produced the highest R^2 value were considered to be the best fit values. This process was repeated for each bar diameter and test temperature considered in the pull-out experiments in chapter 3.

Plots of the best-fit stress-slip curves are included in Appendix A, along with the corresponding experimental stress-slip curves. The best-fit stress-slip curve of the 8mm diameter bar at 110°C is shown here as an example for convenience. The experimental stress strain curve shown in figure 4.3 is the same as the one shown in figure 3.20 for 8mm diameter bars at 110°C.

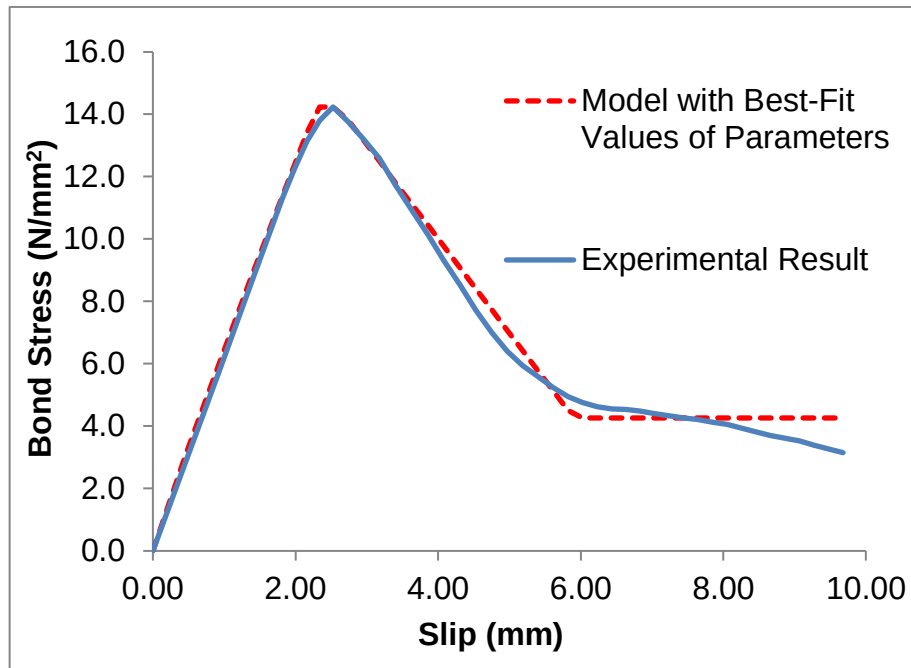


Figure 4.3. Stress-slip curve for 8mm diameter bar at 110°C, calculated from Equations (4.1) to (4.4) with best-fit values of parameters α , s_1 , s_2 , s_3 , $\tau_{b\max}$ and τ_{bf} , with experimental result for comparison

The best-fit values of each of these parameters for each bar diameter and temperature are shown in tables B.1 to B.3 in Appendix B. These best fit values were plotted to identify any distinguishable trend in these values with increasing temperature. The plots of best-fit values of peak bond stress, $\tau_{b\max}$, and the bond stress due to friction, τ_{bf} , with increasing temperature are shown in figures 4.4 and 4.5 respectively. The equivalent plots for the parameters α , s_1 , s_2 and s_3 are included in Appendix C. Figure 4.6 shows the ratio of τ_{bf} to $\tau_{b\max}$ using the best-fit values of this ratio at different temperatures.

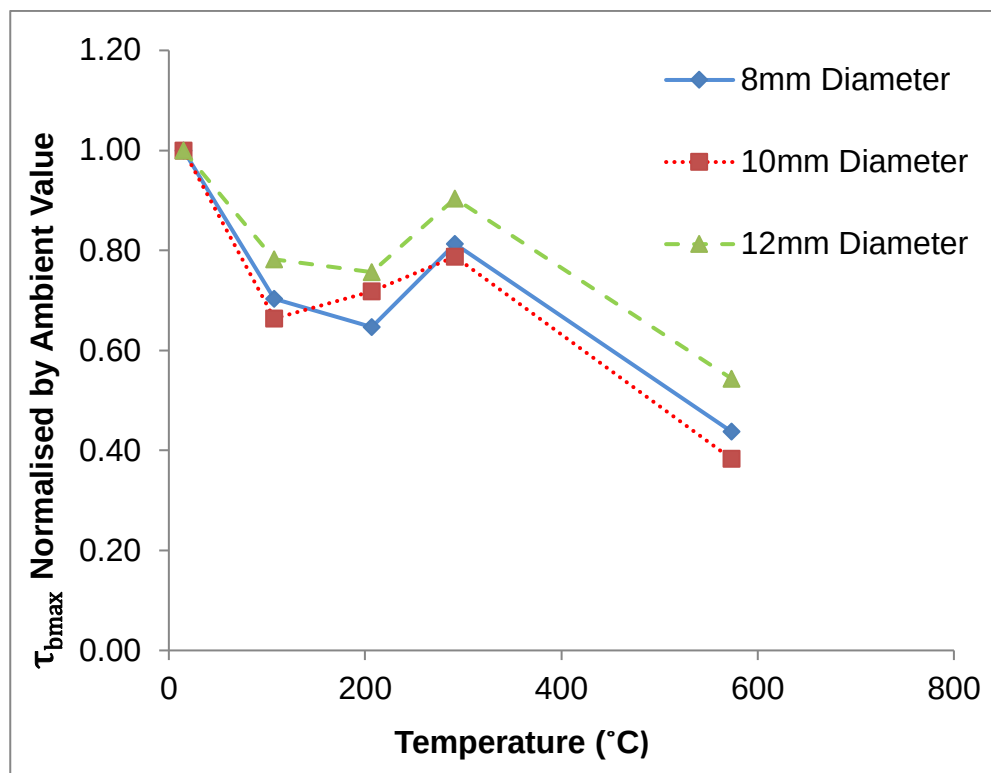


Figure 4.4. Best-fit values of τ_{bmax} at each temperature, normalised by ambient value

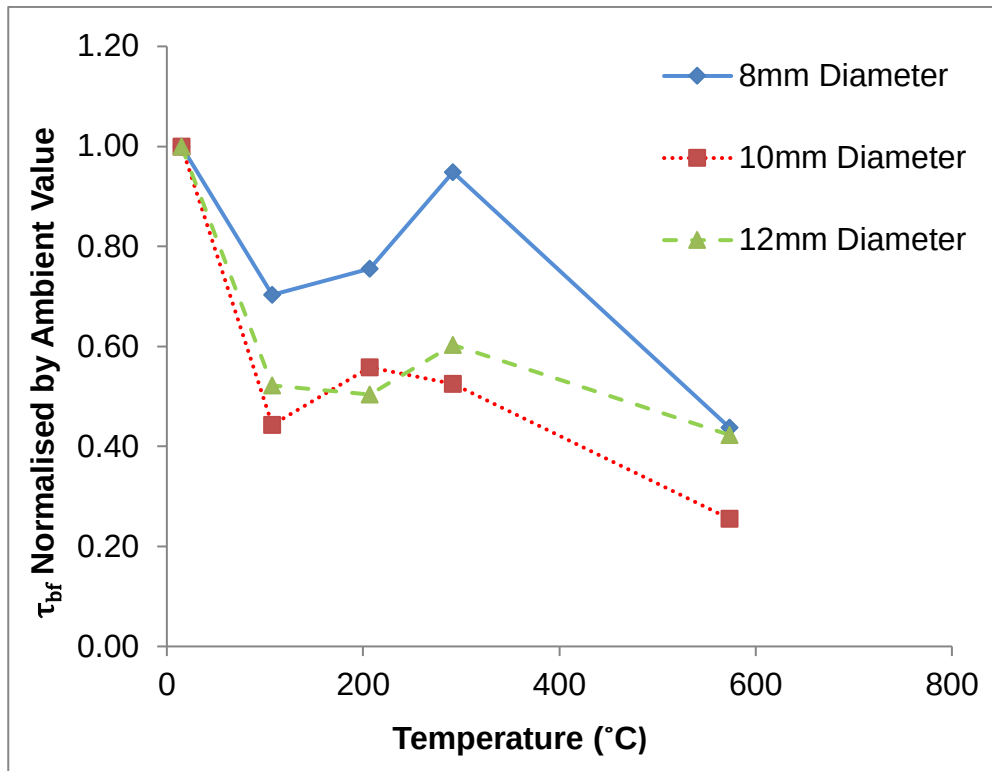


Figure 4.5. Best-fit values of τ_{bf} at each temperature, normalised by ambient value

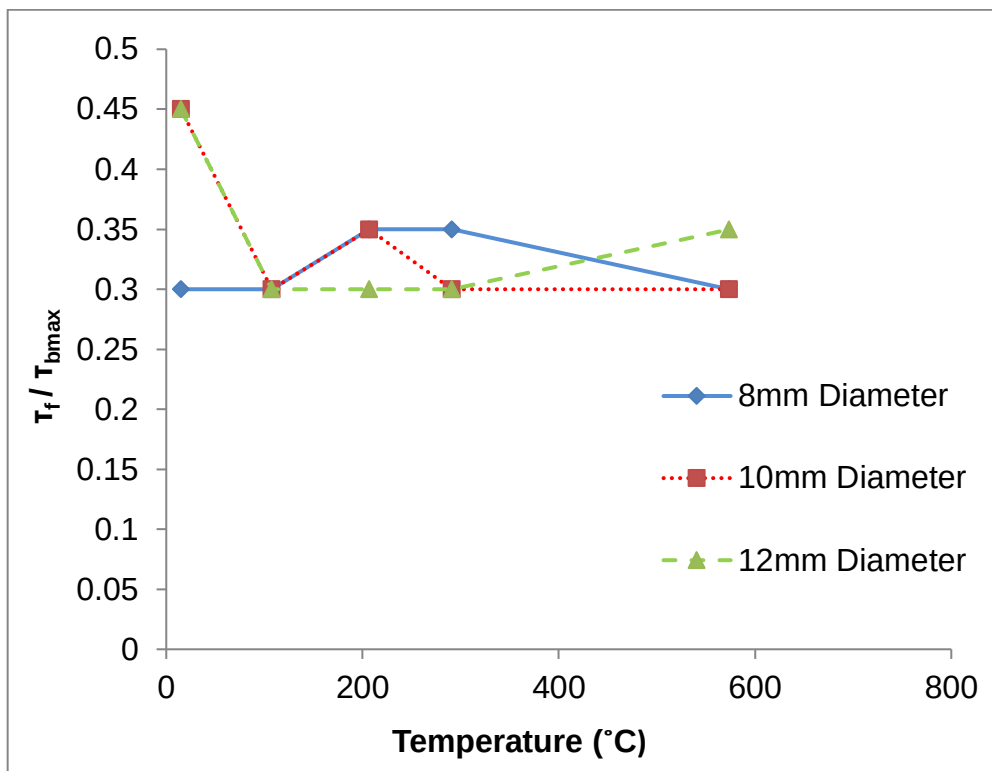


Figure 4.6. Ratio of τ_{bf} to τ_{bmax} using values of best-fit to experimental data

4.3.1 Derivation of bond stress parameters

As evident from figures 4.4 and 4.5, the parameters τ_{bmax} and τ_{bf} follow the down-up-down trend in bond strength which has been discussed previously. Although the values of this ratio in figure 4.6 are typically nearer to 0.35 than 0.4 in the current experimental results, the intention of the research presented in chapter 4 is to develop a modified bond model which accounts for the effects of elevated temperature and there was no clear or strong trend in the ratio of τ_{bf} to τ_{bmax} with increasing temperature. There is, therefore, no basis to recommend that the ratio of τ_{bf} to τ_{bmax} should change with increasing temperatures. It is proposed, therefore, that the proposed model should retain the fib recommended value of 0.4 for the ratio between τ_{bf} to τ_{bmax} . This means that only τ_{bmax} will be directly modified as τ_{bf} is calculated as a proportion of τ_{bmax} .

To modify τ_{bmax} into its temperature dependent form, $\tau_{bmax,\theta}$, a new reduction factor $k_{bond,\theta}$ is applied as per equation (4.9). The bond strength reduction factor $k_{bond,\theta}$ is calculated from equation (4.8). As discussed in section 3.3.2, the change in bond strength retention at elevated temperatures is dependent on the change in concrete compressive strength and these parameters follow a similar trend with increasing temperature. The bond strength reduction factor, $k_{bond,\theta}$, is, therefore, a factor of the compressive strength reduction factor, $k_{c,\theta}$, within the proposed equation 4.8. The values of $k_{c,\theta}$ should be sourced from elevated temperature compressive strength test data.

$$k_{bond,\theta} = \left[1 - \frac{(\theta - 20)}{1500} \right] k_{c,\theta} \quad (4.8 - \text{repeated from section 4.2})$$

$$\tau_{bmax,\theta} = k_{bond,\theta} \cdot \tau_{bmax} \quad (4.9 - \text{repeated from section 4.2})$$

In the derivation of the proposed model, the values of $k_{c,\theta}$ are based in experimental data for elevated temperature compressive strength tests by Castillo and Durrani (1990) for concrete with an ambient compressive strength of 62.8MPa. This source was selected because the test results by Castillo and Durrani (1990) are clearly described, with tests conducted at regular intervals throughout the temperature range being considered in the current research and the tests were conducted at elevated temperature, rather than residual strength tests. The concrete was also of a comparable strength to that used in the pull-out experiments in the current research.

The factor, $k_{c,\theta}$, on its own does not produce an accurate reduction in bond strength. This is because the reduction in bond strength is more severe than the reduction in concrete compressive strength, as discussed in section 3.3.2. A linear reduction with increasing temperature in the form of $\left[1 - \frac{(\theta-20)}{X}\right]$ has been included in equation (4.8). Regression analysis was used to determine the value of X, which was determined to be 1500°C. Figure 4.6 shows the value of R^2 between calculated values of $k_{bond,\theta}$ at different temperatures, using different values of X, and experimental values of $k_{bond,\theta}$, from the results of the pull-out tests conducted in the current research. Although each of the three bar diameters in figure 4.7 gives different values for X, a single value of 1500°C was chosen to represent all three bar diameters.

Each of the three curves in figure 4.7 illustrates the quality of fit which the term $k_{bond,\theta}$ would have to the experimental results when different values of X are used. The different trend in the 12mm bar relative to the 8 and 10mm bars is due to the difference in the trend of bond strength retention at elevated temperature as measured in the experiments in chapter 3. The trend in bond strength retention from experiments is shown in figure 3.24, and it demonstrates a consistently higher

retention of bond strength than the 8 or 10mm bars. There is substantial random scatter in the experimental values and each bond strength value is averaged from a small data set. This limits the level of confidence in the bond strength retention trend for any individual bar diameter, because the entire trend depends heavily on the ambient bond strength measurements. Despite the difference in trends, equation 4.8 with the value of 1500°C as X does give a reasonably good fit for all three bar diameters, as evident from figure 4.7.

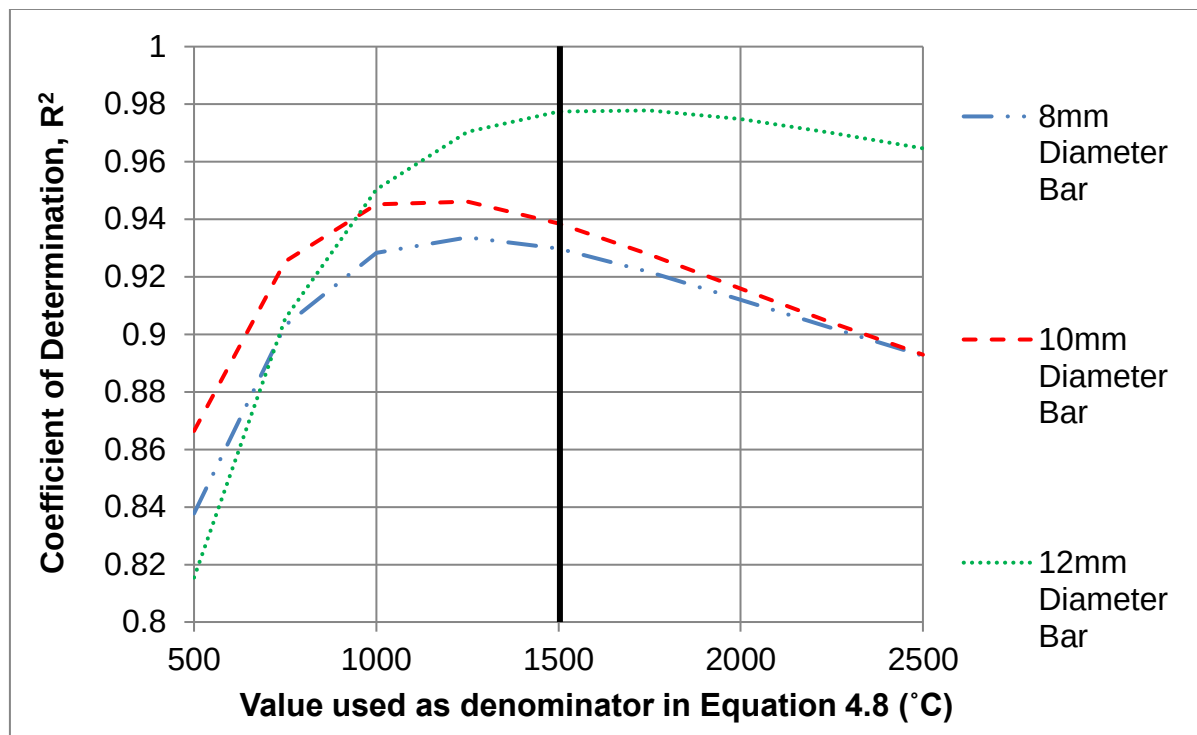


Figure 4.7. Coefficient of determination, R^2 , showing the proportion of variance between the calculated and experimental values of $k_{\text{bond},\theta}$ where different values are adopted as the denominator in Equation 4.8

4.3.2 Derivation of parameters α , s_1 , s_2 and s_3

The plots of best-fit values of the parameters α , s_1 , s_2 and s_3 are included in appendix C. No clear trend could be identified in the parameters α , s_1 , s_2 and s_3 with increasing

temperature, and any effect of temperature on these parameters was small relative to the effect on parameters τ_{bmax} and τ_{bf} . It was, therefore, decided that it would be reasonable to retain the values recommended by fib for the parameters α , s_1 , s_2 and s_3 . For all parameters of the proposed modified model, the applicable temperature range is 20°C to 570°C, and they are applicable for 8, 10 and 12mm diameter deformed bars because these are the ranges of the experimental data which was used to develop the modified model. Further research of bond at elevated temperature would be required to determine the range of applicability of the modified bond model for concrete grades which are different to that used in the experiments in chapter 3.

4.4 Comparison of modified model with experimental results

Stress-slip curves from the modified model have been compared with the experimental results to demonstrate the quality of fit. In these comparisons, parameters s_1 , s_2 , s_3 and α were assigned the values which had been identified from regression analysis as the best-fit values at the ambient temperature for each bar diameter. The ambient temperature experimental values of τ_{bmax} for each bar diameter were used in equation (4.9) in the calculation of $\tau_{bmax,\theta}$ and τ_{bf} . These comparisons demonstrate the ability of the modified model to account for the effects of elevated temperatures.

Figures 4.8 to 4.11 compare the bond stress-slip curves from the modified model with the experimental results for the 8mm diameter deformed bars at each test temperature. Figures D.1 to D.8 in appendix D show the equivalent plots for the 10 and 12mm diameter bars. The proposed model calculates bond stress values for bars of different diameters and temperatures. Additionally, the bond stress values are calculated from compressive strength test results for a different concrete. For these reasons, the quality of fit of the calculated bond stress-slip curves will inevitably have poorer correlation to each experimental stress-slip curve than the curves using best-fit parameters in figure 4.3 and figures A.1 to A.15 in appendix A, which were directly fitted to the experimental curves for each bar diameter and temperature.

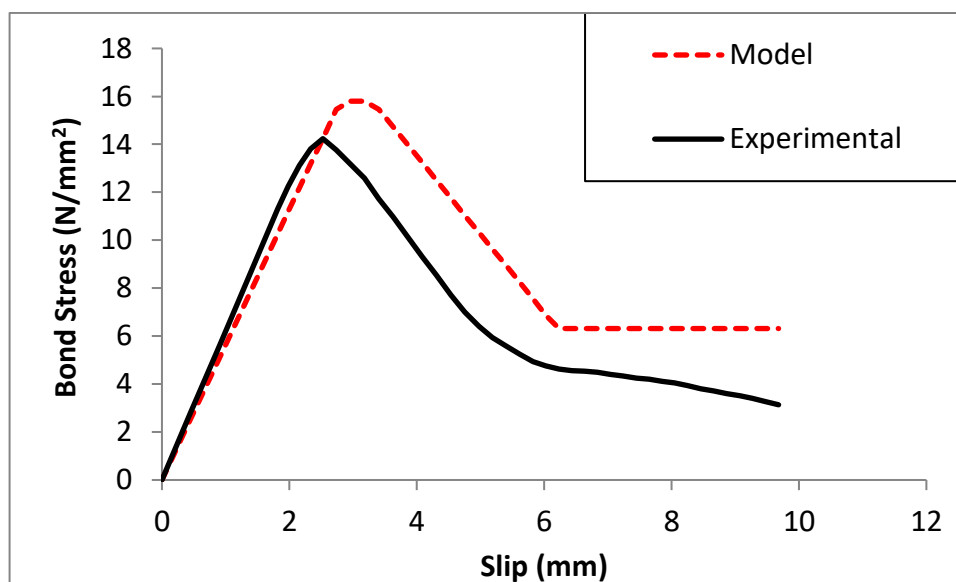


Figure 4.8. Comparison of bond stress-slip of proposed model and experimental results for 8mm bar at 110°C

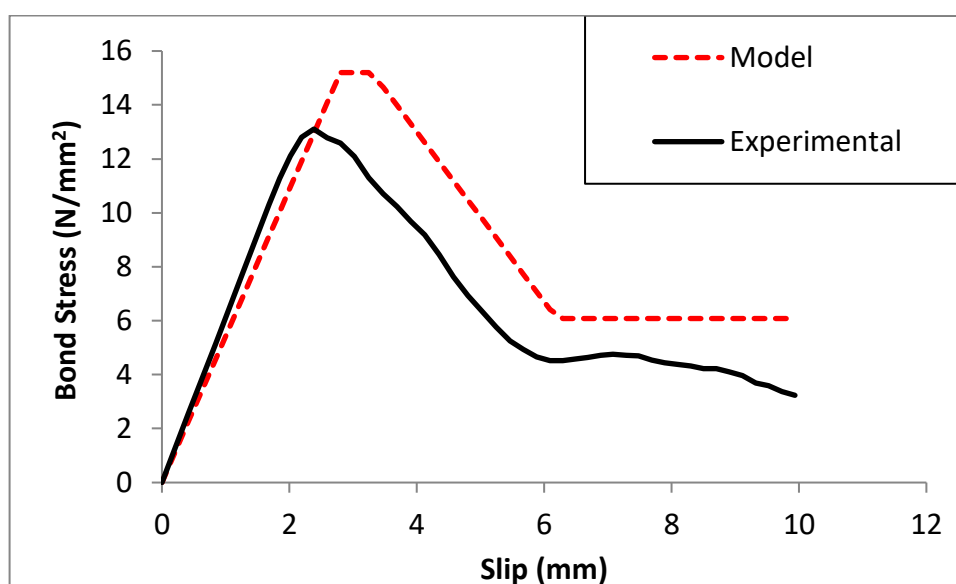


Figure 4.9. Comparison of bond stress-slip of proposed model and experimental results for 8mm bar at 210°C

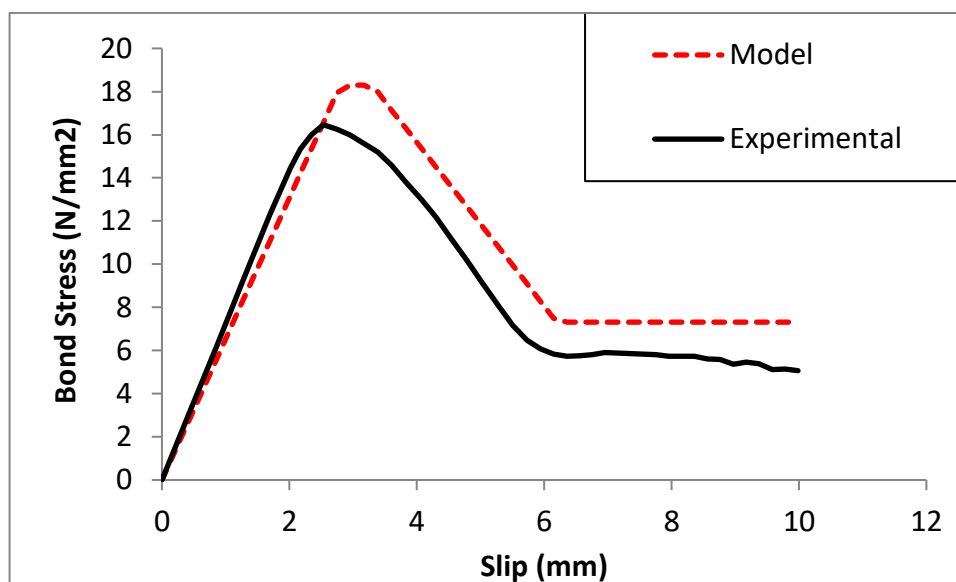


Figure 4.10. Comparison of bond stress-slip of proposed model and experimental results for 8mm bar at 290°C

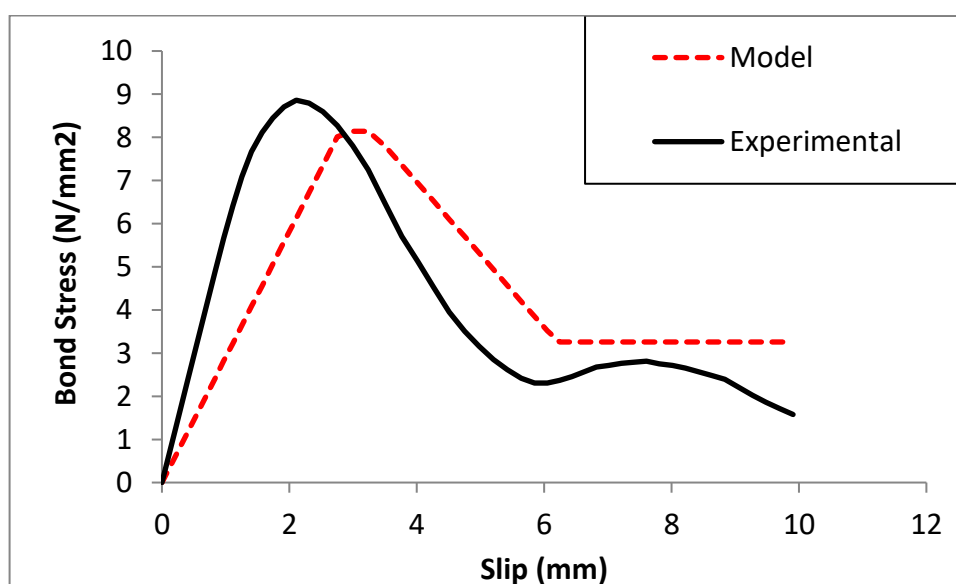


Figure 4.11. Comparison of bond stress-slip of proposed model and experimental results for 8mm bar at 570°C

The percentage difference between the maximum bond stress calculated by the proposed model and the experimental result from chapter 3 for each bar diameter and temperature are shown in table 4.1, with negative values representing underestimates by the model. The model overestimates the bond stress for some bars at some temperatures and underestimates bond stress for other temperatures. This is because it is a single model which is intended to provide the best-fit to results for the three different bar diameters. The fact that the model may overestimate or underestimate bond stress for a given bar or given temperature is a limitation of the model which should be considered when using the model. The accuracy of the model may be reduced for bond with conditions which differ from the experiments in chapter 3, such as with different concrete mixes.

Temperature (°C)	8mm	10mm	12mm
110	11%	18%	0%
210	16%	4%	-1%
290	11%	15%	0%
570	-8%	4%	-26%

Table 4.1. Percentage difference between maximum bonds tress calculated by the proposed model and the experimental result from chapter 3

The model is intended to be a relatively simple model which provides the best fit to the experimental data for all bar diameters and temperatures which were considered in chapter 3. The model is not intended to provide a conservative estimate of bond stress because there are conditions in which overestimating bond strength would be conservative and there are conditions where underestimating bond strength would

be conservative. For example, lower bond strength would generally be preferable for crack-bridging reinforcement, but higher bond strength would generally be preferable for anchorage.

Table 4.2 shows the R^2 values between calculated experimental stress-slip curves. These values correspond to the calculated and experimental stress-slip curves in the figures 4.8 to 4.11 and figures D.1 to D.8 in appendix D.

Temperature (°C)	8mm	10mm	12mm
110	0.84	0.52	0.85
210	0.83	0.98	0.75
290	0.92	0.97	0.97
570	0.48	0.57	0.95

Table 4.2. Coefficient of determination, R^2 , of stress-slip curves from test data vs. calculated values

Figures 4.12 to 4.14 provide a comparison of the experimental and calculated values of maximum bond stress at different temperatures for each bar diameter. Figures 4.12 to 4.14 compare the maximum bond stress values calculated by equation 4.8 with the values from the experimental results for 8, 10 and 12mm diameter deformed bars. Table 4.2 and the figures shown in this section, along with those in appendix D, together demonstrate an acceptable quality of fit between the proposed model and the experimental results, given the level of scatter in the experimental results and the lack of information on the compressive strength of the concrete actually used at elevated temperature.

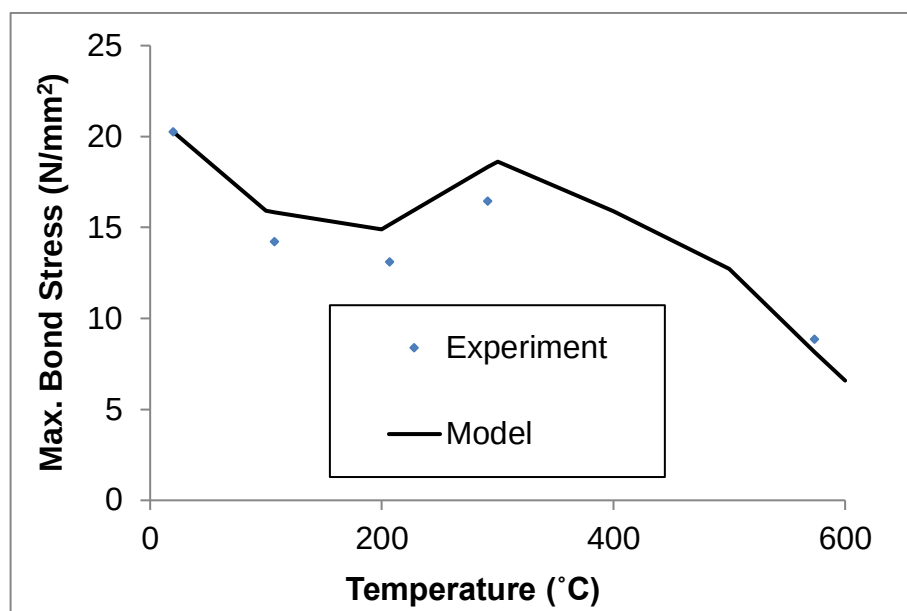


Figure 4.12. Comparison of maximum bond stress from proposed model and experimental results for 8mm diameter bar.

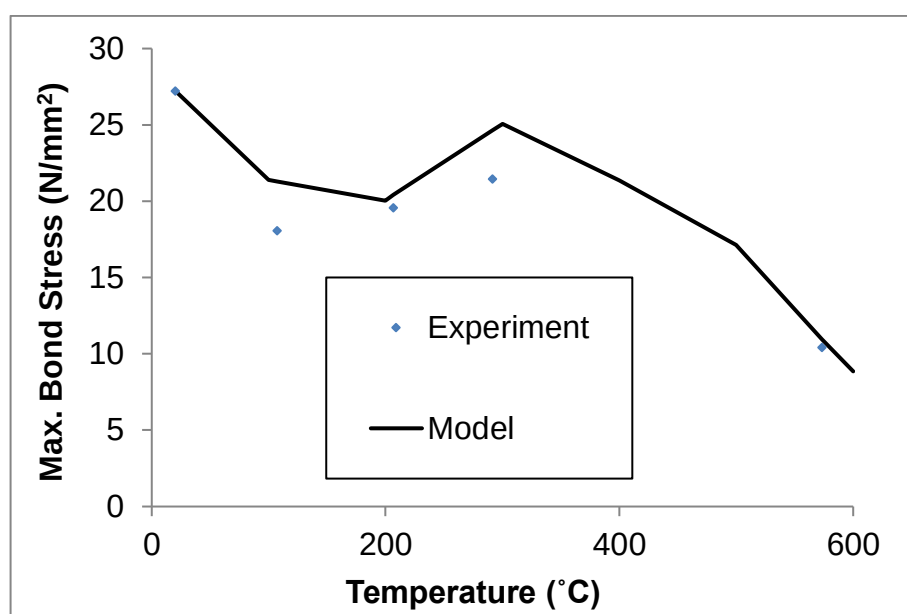


Figure 4.13. Comparison of maximum bond stress from proposed model and experimental results for 10mm diameter bar

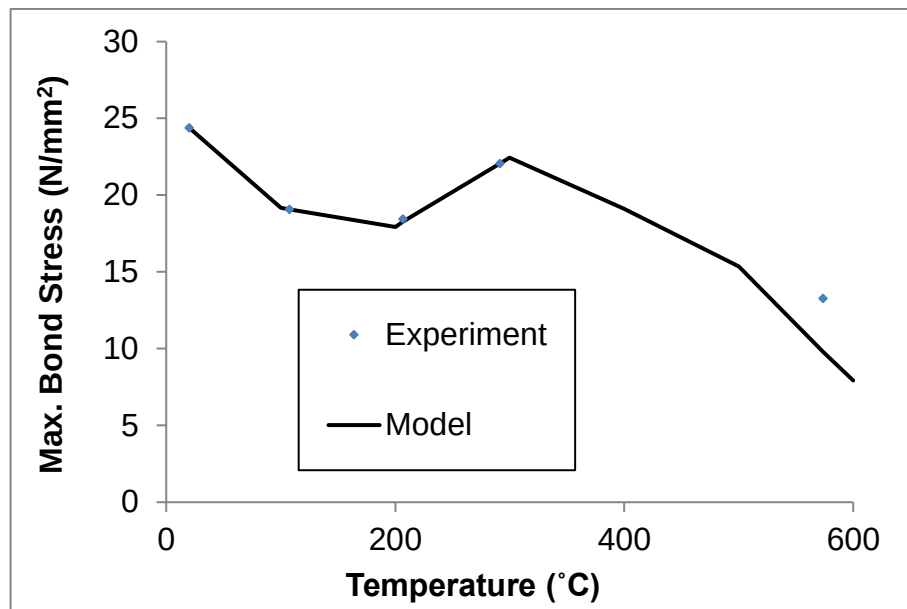


Figure 4.14. Comparison of maximum bond stress from proposed model and experimental results for 12mm diameter bar

4.5 Comparison of modified bond model with previous works

Values of the proposed bond strength elevated temperature reduction factor $k_{\text{bond},\theta}$ have been calculated for comparison with the experimental results of elevated temperature pull-out tests by Morley and Royles (1983). As the compressive strength reduction factor, $k_{c,\theta}$, is required in the calculation of $k_{\text{bond},\theta}$, a suitable source of values of $k_{c,\theta}$ had to be selected. For the purposes of this comparison, two sets of values of $k_{c,\theta}$ have been used. The values of $k_{c,\theta}$ are based on compressive strength tests by Castillo and Durrani (1990) and compressive strength tests by Morley (1982) which were conducted in the same study as the bond tests. The compressive strength tests by Castillo and Durrani (1990) were conducted at elevated temperature and those by Morley (1982) were residual strength tests, conducted after cooling. Therefore, neither set of test data represents the strength of the concrete used at the temperature which the bond tests were conducted.

Figure 4.15 compares the bond strength retention calculated from the proposed factor $k_{\text{bond},\theta}$ with the bond strength retention of a 16mm deformed bar tested at elevated temperature by Morley and Royles (1983). The quality of fit is as good as can be expected given the lack of information on the strength of the particular concrete used at these temperatures.

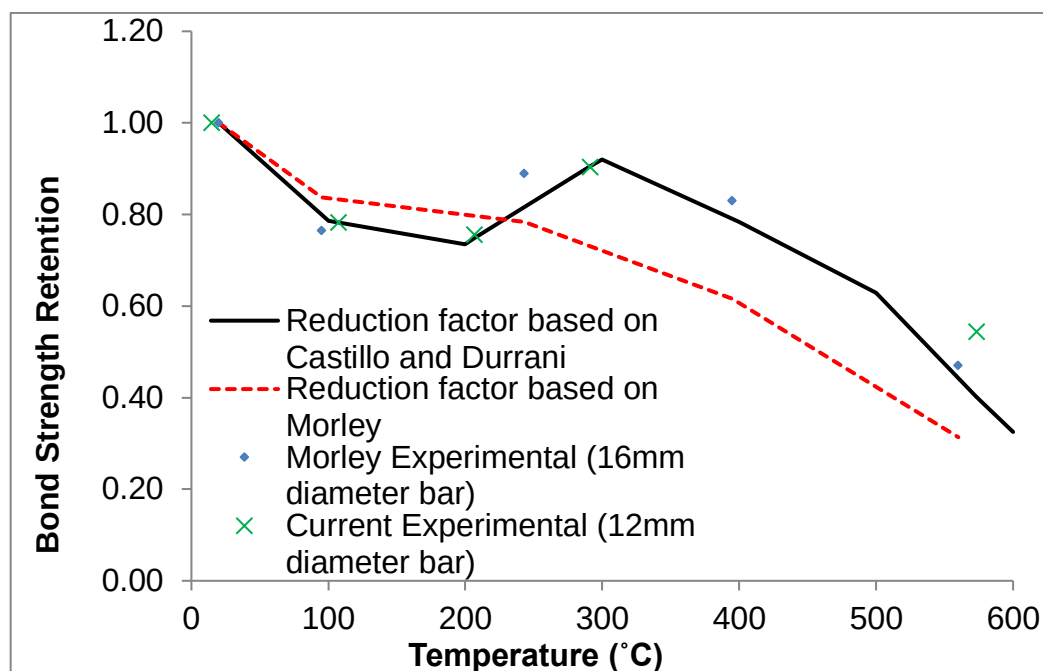


Figure 4.15. Comparison between bond retention from Morley and Royles (1983) experimental results for 16mm diameter deformed bar, current experimental results for 12mm diameter bar and proposed factor $k_{\text{bond},\theta}$ with $k_{c,\theta}$ based on studies by Castillo and Durrani (1990) and Morley (1982)

4.6 Further development and the applicability of the modified bond model

The fib local bond model is intended for use at ambient temperature, and does not accurately account for the effects of elevated temperature. The proposed modified model is useful as it allows bond stress and slip behaviour to be calculated at elevated temperatures with greater accuracy because it has been derived from experimental stress-slip data of bond tests at elevated temperature. This model will be useful in numerical modelling as the local bond model can be extrapolated for use with longer bonded lengths. The model can be utilised in research studying elevated temperature bond behaviour, anchorage and crack-bridging of reinforcement.

The model is derived from limited test results and should be considered in this context. It would be beneficial to compare the model to further test data using modern construction materials if such data becomes available. It would also be advantageous to expand the scope of the model for use with larger bar diameters if suitable data becomes available to derive these relationships.

The selection of appropriate values for the parameter $k_{c,\theta}$ from test data is critical to the accuracy of this model. The values of parameter $k_{c,\theta}$ must be compatible in terms of mix design and compressive strength. Comprehensive testing of concrete compressive strength at elevated temperatures considering a range of modern concrete mixes would improve understanding and realism in the modelling of bond of deformed bars at elevated temperatures.

5 Numerical modelling of reinforcement behaviour in fire

5.1 Introduction

The reason for developing this finite element (FE) model is to investigate the behaviour of reinforcement in composite or reinforced concrete slabs with continuity over perimeter supporting beams in the situation where tensile membrane action develops under fire conditions. This condition is described in section 2.3. It is unknown how modern deformed reinforcement will behave in this situation, and whether the reinforcement will be prone to rupture at the location of large cracks around the perimeter of the slab as tensile membrane action develops. The rupture of the reinforcement could result in structural collapse and, therefore, this behaviour must be investigated.

An FE model has been developed using Abaqus CAE software. The FE model represents a region of interest within a composite slab, but is simplified as a reinforced concrete slab of flat profile. The region of interest is located over a vertical support, where severe cracking would develop during the development of tensile membrane action. An FE modelling approach allows details of strain and bond slip, which would be difficult to observe in an experiment with reinforced concrete at elevated temperatures, to be studied.

The specimen which is described in chapter 6 was chosen to allow the crack-bridging reinforcement to be studied, rather than representing a particular slab. The slab is quite under-reinforced, but this is deliberate, as it is cracking behaviour that is

being investigated and not concrete crushing failure. The FE model simulates the cracking of the slab through the concrete damage plasticity function in Abaqus.

Reinforcement is represented in the FE model by solid elements and the bond of the reinforcement is represented numerically as a temperature-dependent cohesive interaction between surfaces at the interface. The susceptibility of the reinforcement to rupturing and the ability of the slab to act under tensile membrane action without reinforcement rupture under different conditions has been investigated. Conditions of fire duration, temperature, bar surface type, bar diameter and slab depth have been considered.

A sensitivity analysis was conducted on mesh size to confirm the suitability of the modelling approach. The bond modelling approach has been validated by using the same overall approach to model the pull-out experiments and comparing the results in terms of stress and slip at elevated temperature. The overall modelling approach has also been validated by using the same approach to model a beam experiment conducted by others and comparing the results in terms of the load-deflection response. The mesh sensitivity analysis is included in section 7.2, after an initial explanation of the results to provide necessary context to the level of sensitivity. Chapter 5 explains the numerical modelling methodology. Sections 5.5 and 5.6 present validation studies of the numerical modelling methodology. Figure 5.1 provides an overview of the numerical modelling research which is presented in chapters 5 to 7.

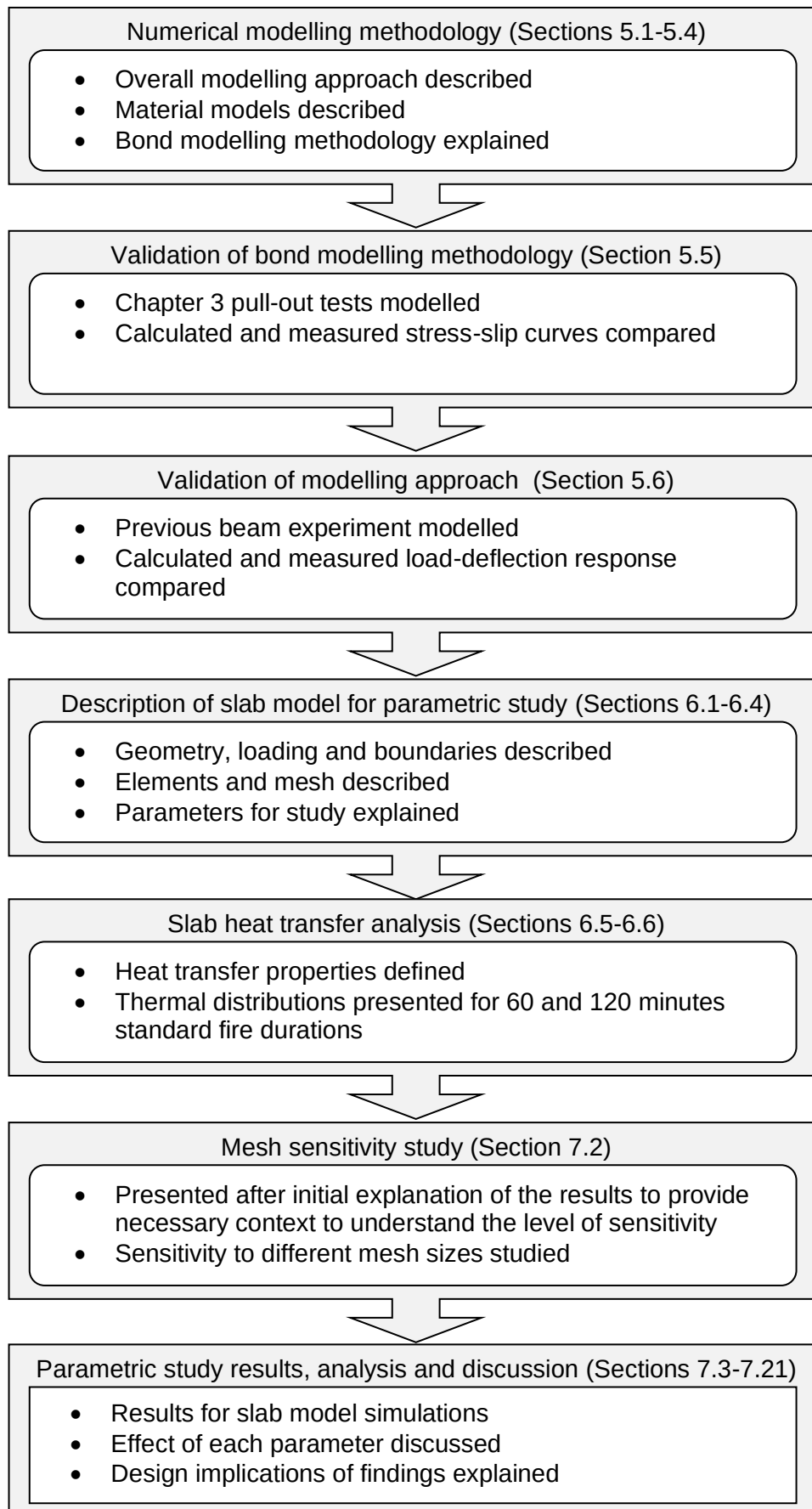


Figure 5.1. Overview of the numerical modelling research

5.2 Overview of numerical modelling methodology

The reinforcement bar and the concrete of the slab specimen were modelled using solid elements. Concrete damage and cracking were represented using the concrete damage plasticity model in Abaqus. The damage and rupture of the steel were represented using the ductile damage model in Abaqus. More detail on the modelling of these materials is provided in section 5.3.

The interaction between the bar and concrete has been modelled using a general contact definition between surface pairs, using the cohesive type of interaction in Abaqus. The definition of the cohesive interaction for deformed bars was based on the calculated bond stress-slip curves in appendix D. The inputs are, therefore, calculated from the newly proposed equations 4.8 and 4.9 in chapter 4. Values of s_1, s_2, s_3 and α , which define slip values and the shape of the initial curve of the bond stress-slip response, were based on the values which were found to be of best-fit to the ambient stress-slip curve for each bar diameter. The inputs for the bond interaction for the plain round bars are directly calculated from the experimental stress-slip curves in figure 3.26, as the fib model is not applicable to plain round bars.

Comparing the response of plain round and deformed bars is a key focus of this research. It was, therefore, important that the input sources were comparable for both bar types. The bond stress-slip curve which was used to define the bond interaction for the plain round bars was taken directly from experimental results and the bond stress-slip curve which defined the bond interaction for the deformed bars was taken from bond stress-slip curves that were fitted to experimental results. This

is explained further in section 5.4. Although it is appropriate to use the values recommended by fib for s_1, s_2, s_3 and α as a straightforward and generalised approach, the values of best-fit to the experimental data were preferred in this instance because they are more comparable to the experimental data which was used for plain bars.

The choice of a three dimensional model, rather than two dimensional, was due to the use of cohesive interactions, which are a type of surface to surface interaction, to represent the bond stress-slip interaction. This type of interaction is assigned to surfaces and required the opposing concrete and steel surfaces to be modelled, and to each have a surface area on which the traction between the two surfaces could be defined on a per-unit-area basis. This would not have worked in two dimensions because the opposing steel and concrete surfaces would have no area. A two dimensional approach using connector elements was initially considered during model development, but the cohesive interaction approach appeared to have better stability and was less time consuming to assign. In principle, apart from the interaction modelling considerations, a two dimensional approach would have been a reasonable simplification which could produce similar outcomes if modelled correctly.

Hexahedral elements were used for both concrete and steel. For three dimensional analysis, hexahedral elements generally give good performance and low computational cost (Dassault Systèmes, 2012). The steel elements had the element type C3D8, which is a general purpose linear brick element and was adequate and efficient. The concrete elements were assigned the option of improved surface stress visualisation, with the element type C3D8S. This element type provides a superior stress visualisation as it avoids errors from extrapolation of stress components

because the elements have integration points at the nodes (Dassault Systèmes, 2012).

Analysis was performed with Abaqus/Standard, using the 'full Newton' solution technique and the direct method equation solver. The structural analysis was performed in a 'static, general' procedure. The heat transfer analysis is described in section 6.5.

A mesh sensitivity study was conducted on the slab model and is described in section 7.2. This mesh sensitivity study demonstrated that the mesh size used was suitable for the slab model. Given the similarity of the slab model to the other models used in this research, which have identical bond interaction assignments and the same typical mesh sizes, the mesh sensitivity study on the slab model is considered sufficient to demonstrate the suitability of the mesh size for all of the models which were used in this research.

5.3 Material definitions

5.3.1 Concrete

Concrete has been modelled using the concrete damage plasticity model within Abaqus. Concrete damage plasticity was used as it allows the cracking and crushing behaviour of concrete to be represented. The Abaqus concrete damage plasticity model was described in section 2.7. The values used to define concrete material in the current FE model are described in this section.

The ratio of initial equibiaxial to initial uniaxial compressive yield stress (σ_{b0}/σ_{c0}) has been assigned the value of 1.16, which is the default value in Abaqus, as this agrees with the findings of Monte, Kalaba and Bamonte (2017). The parameter K_c , which is

the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian, was given the Abaqus default value of $2/3$, based on the findings of Monte, Kalaba and Bamonte for temperatures under 600°C (2017). The eccentricity parameter, (ϵ) , was given the Abaqus default value 0.1 . The dilation angle parameter, ψ , was given the value of 35° , based on the findings of Wahid *et al.* (2019). The viscosity parameter, μ , was assigned the value of $0.0001t$, based on the recommendation of Wahid *et al.* (2019). The viscosity parameter enabled viscoplastic regularisation of the concrete constitutive equations. A value which is small, relative to the characteristic time increment, can improve the rate of convergence without compromising results (Dassault Systèmes, 2012).

The temperature dependent compressive behaviour of concrete was defined in Abaqus in tabular format in terms of compressive stress (f_c) and inelastic strain ($\epsilon_{c,in}$) with the curve as represented by figure 5.2, where $f_{c,T}$ is the characteristic compressive strength of the concrete at a given temperature (Dassault Systèmes, 2012). The values of the stress-strain curves for each temperature are according to Eurocode 4 Part 1-2 (BSI, 2005b). The elastic modulus ($E_{o,T}$) at each temperature was taken as the gradient from the origin to the point $0.4f_{c,T}$, consistent with the secant modulus definition in Eurocode 2 (BSI, 2019).

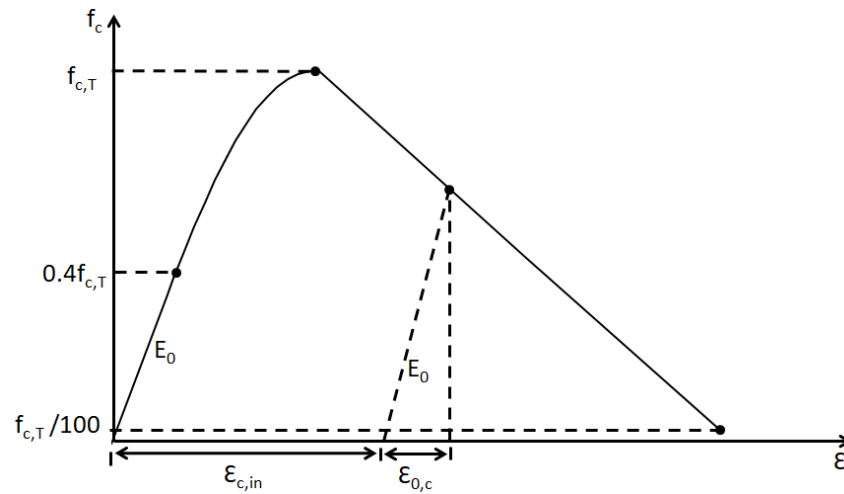


Figure 5.2. Definition of concrete compressive stress-strain data, based on Dassault Systèmes (2012)

Ambient tensile strength of the concrete was estimated from the characteristic compressive strength using the following equation 5.1, taken from fib (International Federation for Structural Concrete, 2013), for concrete not greater than C50.

$$f_{ctm} = 0.3 \cdot (f_{ck})^{2/3} \quad (5.1)$$

The ambient characteristic compressive strength of concrete, f_{ck} , was taken as 42MPa, as this value is well within the typical range of concrete in construction. The model of concrete tensile strength retention at elevated temperatures proposed by Aslani and Bastami (2011) was adopted in this study. The softening response adopted in this study was the bilinear softening model by Petersson (1981), as shown in figure 5.3. In figure 5.3, $f_{t,T}$ is the tensile strength of the concrete at temperature T , ε_t^{ck} is the concrete cracking strain and $\varepsilon_{0,t}$ is the elastic strain corresponding to the undamaged material.

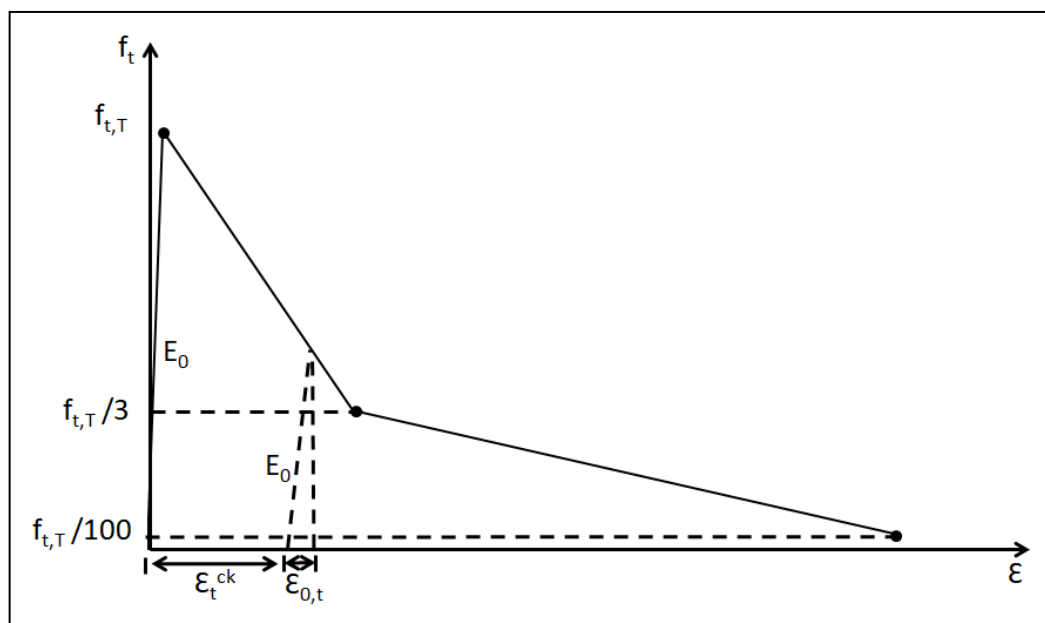


Figure 5.3. Definition of concrete tensile stress-strain data, based on the softening response by Petersson (1981)

Fracture energy, G_F , was calculated according to the method given by the International Federation for Structural Concrete (1993), with the value of G_{f0} taken as 0.026N/mm based on a maximum aggregate size of 10mm. Tensile cracking strains were defined as cracking displacements divided by the characteristic length of the elements. For the 3D elements used in this model, the characteristic length can be defined as the cubic root of the volume of the element.

Temperature dependent values of Poisson's ratio for concrete were taken from an experimental study by Barh *et al.* (2012). The measurement of Poisson's ratio was unreliable above 500°C, so the value has been taken as constant from 500°C with increasing temperature.

5.3.2 Steel

The stress-strain relationship of the steel at each temperature was that of structural steels in Eurocode 4 Part 1-2 (BSI, 2005b) but with an ambient yield strength of

500N/mm² for deformed bar and 250N/mm² for the plain round bars. The curves were used as they have clearly defined critical strains and they are temperature dependent. The appropriateness and limitations of these curves, in terms of ductility, have been taken under consideration in interpretation of the model outputs.

This relationship includes strain hardening effects in the range between the yield strain, $\varepsilon_{ay,\theta}$, and the limiting strain for yield strength in the fire situation, $\varepsilon_{au,\theta}$, at each temperature. The benefit of strain hardening was limited to a factor of 1.1 as recommended by Eurocode 4 Part 1-2 (BSI, 2005b) for reinforcing steel. This factor is the ratio of ultimate tensile strength of the steel in a fire situation, $f_{au,\theta}$, to the yield strength of the steel in a fire situation, $f_{ay,\theta}$. In accordance with Eurocode 4 Part 1-2 (BSI, 2005b), yield strain in the fire situation, $\varepsilon_{ay,\theta}$, is 0.02, limiting strain for yield strength in the fire situation, $\varepsilon_{au,\theta}$, is 0.15 and ultimate strain in the fire situation, $\varepsilon_{ae,\theta}$, is 0.2. These are the strain values which were used as a base case for simulations, and they are shown in figure 5.4. Ductility is a parameter which was investigated in some simulations which are explained in chapter 6, and these strain values were changed for some of the simulations, as explained in section 6.4.

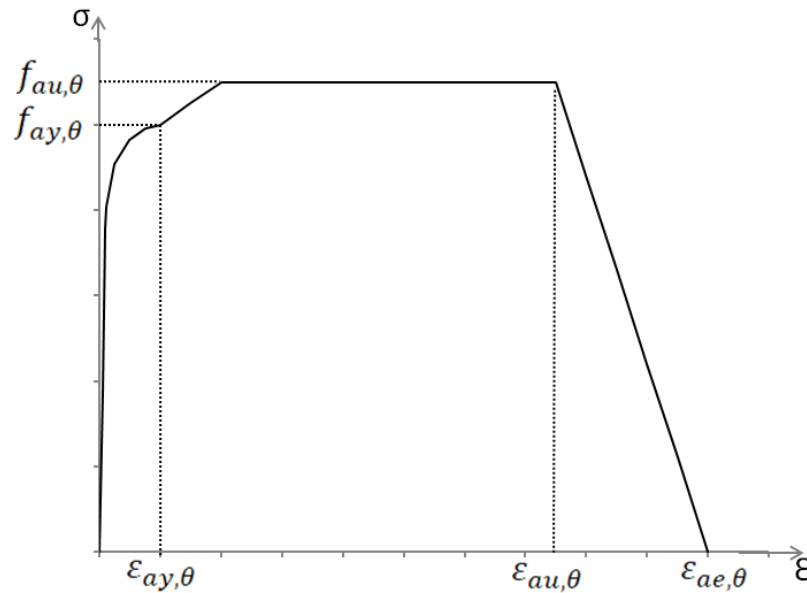


Figure 5.4. Steel stress strain curve based on Eurocode 4 Part 1-2 (BSI, 2005b)

The ambient temperature elastic modulus of steel was taken as 210GPa, as per Eurocode 3 (BSI, 2005a). Temperature dependent values of elastic modulus were used in the Abaqus model. These values are based on a reduction factor for elevated temperatures, taken from Eurocode 4 Part 1-2 (BSI, 2005b). The temperature dependent elastic modulus of steel in the relevant temperature range is shown in figure 2.28.

The ductile damage feature of Abaqus was used to represent the deterioration in strength and stiffness of steel after damage initiation criteria are met. A linear evolution of damage, d , was assumed, as shown in figure 5.4, with the steel stress-strain curve from Eurocode 4 Part 1-2 (BSI, 2005b) including strain hardening effects at elevated temperature.

Stiffness damage was onset at $\epsilon_{au,\theta}$, which is 0.15 from Eurocode 4 Part 1-2 (BSI, 2005b). This strain was converted to the equivalent plastic strain at the onset of damage, $\bar{\epsilon}_0^{pl}$. The damage then increased linearly to $\epsilon_{ae,\theta}$, which is 0.2 from

Eurocode 4 Part 1-2 (BSI, 2005b). Strain values were converted into equivalent plastic strains for input in Abaqus. The equivalent plastic strain at the point of failure, $\bar{\epsilon}_f^{pl}$, in figure 5.5 was taken as the ultimate strain in the fire situation, $\epsilon_{ae,\theta}$, from Eurocode 4 Part 1-2 (BSI, 2005b). This point was defined within Abaqus as a 'displacement at failure' input of 0.0002 based on the strain of 0.2 and a characteristic element length of 1mm. The elements within the area of the bar which ruptures during simulations are not all identical, and the characteristic length is an approximation for the range of elements. The fact that this is an approximation is less significant as the results were interpreted based on the stage at which strain in the bar reached the ultimate strain $\epsilon_{ae,\theta}$, rather than the increment in which rupture was simulated.

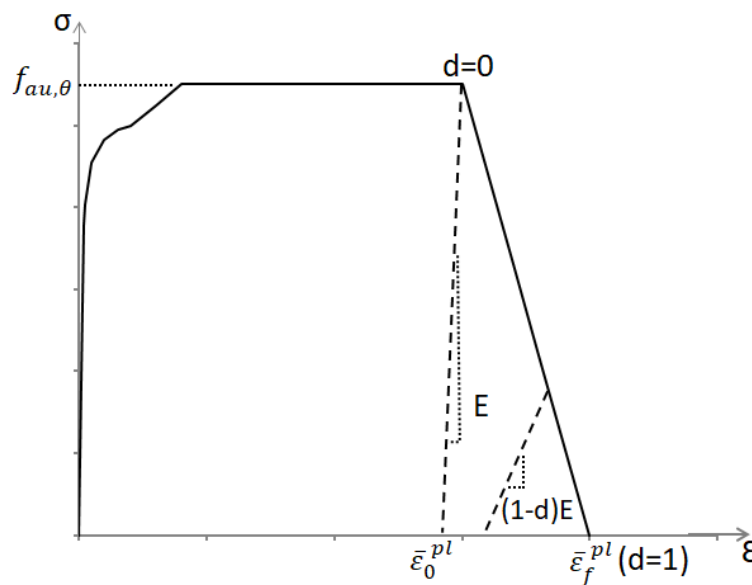


Figure 5.5. Steel stress-strain curve, based on Eurocode 4 Part 1-2, with strain hardening effects and showing the evolution of damage

5.4 Bond interaction

The interaction between the bar and concrete has been modelled using an interaction assignment in Abaqus. The contact properties feature both 'normal' behaviour and 'cohesive' behaviour assignments. The 'normal' contact is defined as 'hard' contact, which is a contact pressure-overclosure relationship which minimises penetration of the slave and master surfaces (Dassault Systèmes, 2012).

When the 'cohesive' interaction property is used, Abaqus follows a linear elastic traction-separation relationship until the onset of stiffness damage. By defining the criteria for stiffness damage and including a tabular definition of stiffness damage values, both in terms of separation, it was possible to define the bond stress-slip relationship to fit any curve. This approach is explained further in this section.

The bond stress-slip curve values for the deformed bars are taken from the stress-slip curves in appendix D. The stress-slip curve values for the plain round bars are directly from the stress-slip curves in figure 3.26. The choice of stress-slip curves was explained in section 5.2. The curves were incorporated through the definition of initial stiffness values, a stiffness damage onset criteria and a table of stiffness damage values with respect to separation values.

The bond interaction is achieved using temperature-dependent stiffness couplings between the separation vectors and the traction vectors contained in an elastic constitutive matrix; as per equation 5.2. The nominal tractor stress vector, $\mathbf{t}$, contains components t_n , t_s and t_t representing the tractions in the local 1, 2 and 3 directions respectively. In this model, t_n acts in the normal or perpendicular direction to the bar surface, t_s acts along the length of the bar surface in the orientation of bond slip and t_t acts in the transverse direction across the bar surface. The corresponding

separations are δ_n , δ_s and δ_t . The stiffness, K , terms in equation 5.2 provide coupled behaviour between denoted components of the traction vector and separation vector.

$$t = \begin{Bmatrix} t_n \\ t_s \\ t_t \end{Bmatrix} = \begin{bmatrix} K_{nn} & K_{ns} & K_{nt} \\ K_{ns} & K_{ss} & K_{st} \\ K_{nt} & K_{st} & K_{tt} \end{bmatrix} \begin{Bmatrix} \delta_n \\ \delta_s \\ \delta_t \end{Bmatrix} = K\delta \quad (5.2)$$

The normal stress-slip and bond stress-slip components were represented by temperature dependent stiffness terms K_{ns} and K_{ss} respectively. Temperature dependent stiffness values K_{ss} were taken as the maximum secant stiffness in the stress slip curves in appendix D.

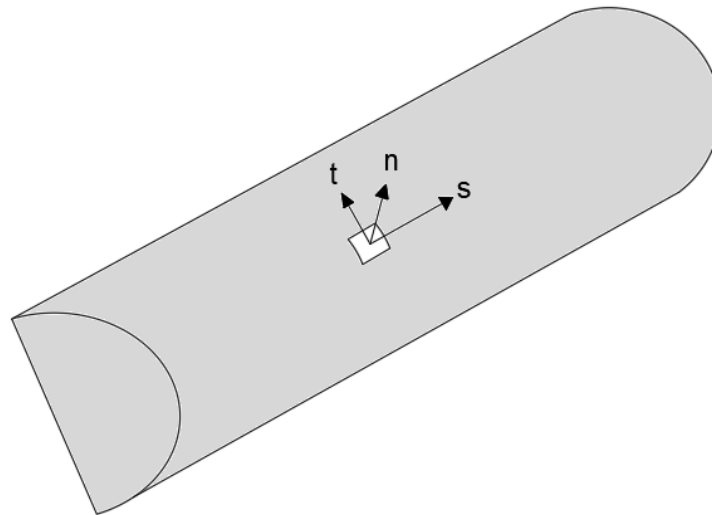


Figure 5.6. Local axis convention for each individual element face on the surface of the reinforcing bar

When slip s_1 , in figure 5.7, is reached cohesive stiffness damage is onset and the bond stiffness begins to reduce. The evolution of this damage was defined with respect to slip. Figure 5.7 and the equations 5.3 and 5.4 explain the manner in which the values of K_{ss} and stiffness damage values were calculated.

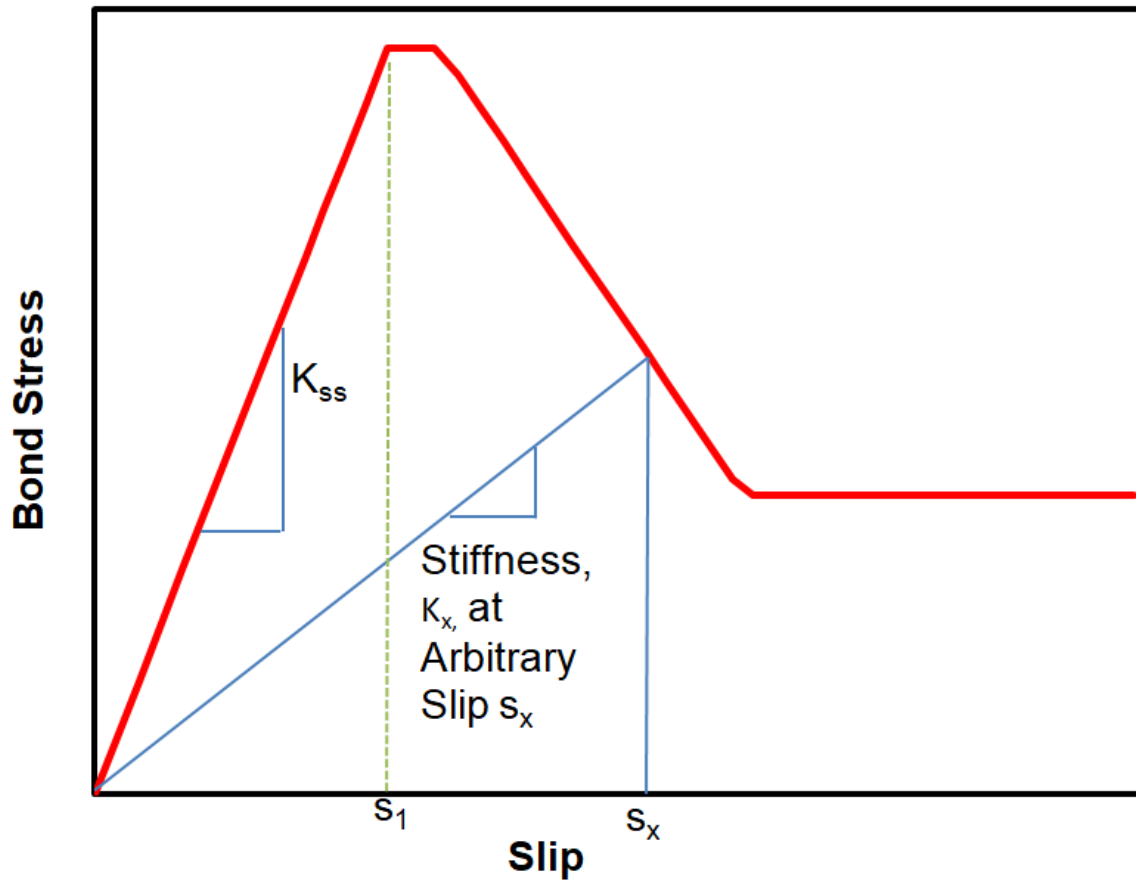


Figure 5.7. Indicative bond stress-slip curve denoting stiffness values of K_x

The damage displacement, μ_x , at an arbitrary slip value s_x is as follows:

$$\mu_x = s_x - s_1 \quad \text{where } s_1 < s_x \quad (5.3)$$

and the damage, d_x , at damage displacement μ_x is calculated as:

$$d_x = \frac{K_{ss} - K_x}{K_{ss}} \quad (5.4)$$

The stiffness component, K_{ns} , is the stiffness which couples the normal stress and the axial slip of the bar. This component was used to represent the effects of the wedging action of the deformed bars. The value of K_{ns} was taken as K_{ss} multiplied by $\cot(25^\circ)$, representing a 25° bearing angle, as was found to be the most probable bearing angle in a study by Han *et al.* (2019).

5.5 Validation of bond modelling method

To confirm the adequacy of the bond modelling approach, an Abaqus model of the pull-out experiments was developed. The bond modelling approach here, and the inputs used for each bar type and diameter, are identical to those used in all of the FE models in the current research, except that the pull-out specimens from chapter 3 are modelled in this section and not a slab or beam.

Simulations were run at ambient and elevated temperature to allow comparison with the equivalent experimental results. Uniform temperatures were assigned for each scenario using predefined field definitions. The pull-out model was created with an 8mm diameter reinforcing bar. Scenarios were modelled with interactions representing both plain and deformed reinforcing bar.

All models in this research used the same typical mesh sizing with a typical mesh size of 2.5mm for the finer regions. The 2.5mm mesh was assigned in the regions of interest around the bond interface. Figure 5.8 shows the meshed model geometry, for the purposes of verifying the bond modelling method. As with all models in this research, some elements were finer than these typical sizes due to the curved shape of the bar.

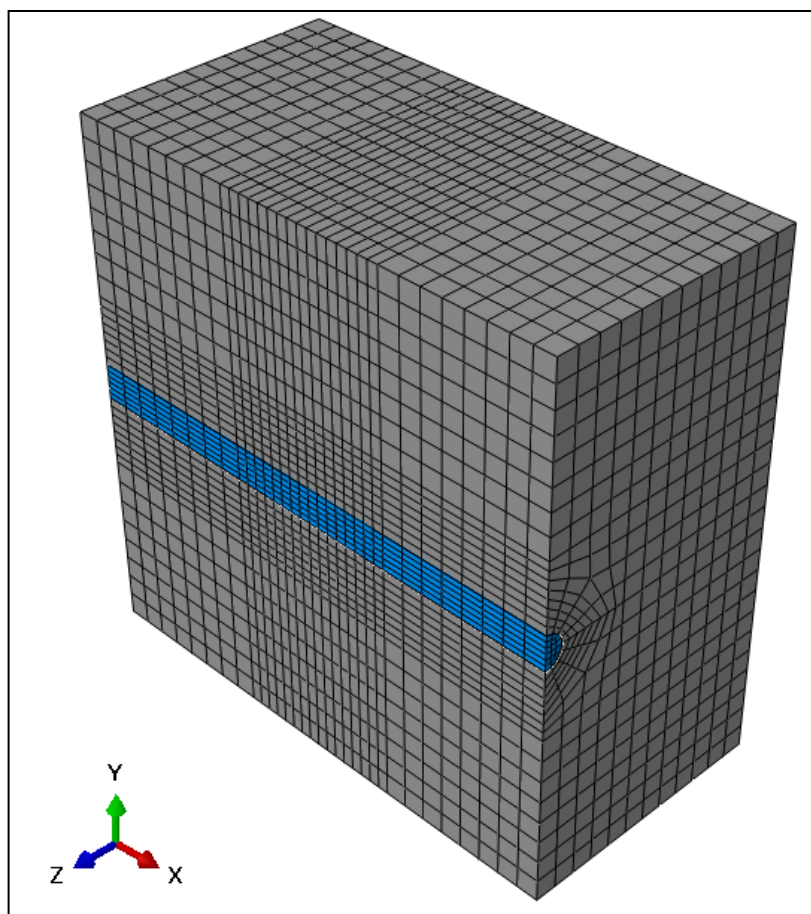


Figure 5.8. Meshed geometry of Abaqus model of pull-out specimen

The modelling approach and property definitions were identical to those used for the slab models except where explained otherwise in this section. The geometry of the pull-out model represented one half of an experimental specimen, with a symmetry boundary condition assigned to the cut surface. The bearing surface of the concrete cube, highlighted red in figure 5.9, was assigned a fixed, or encastre, boundary condition.

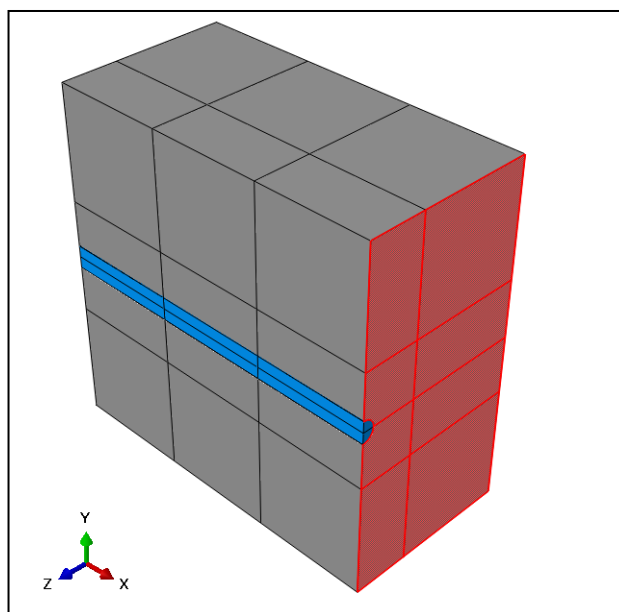


Figure 5.9. View of Abaqus model of pull-out specimen with bearing surface highlighted in red

The surface of the bar outlined red in figure 5.10 was assigned a controlled displacement (+X) boundary condition to induce pull-out of the bar. It is not expected that the displacement rate of 2mm per minute would have induced significant dynamic effects in the pull-out experiment, so it is considered reasonable that a 'static, general' step is used in the analysis. Representing the limited bond length of the pull-out specimens, only the middle 40mm of the bar length was assigned contact interactions with the concrete part. This matches the geometry of the concrete cube and bonded length used in the pull-out tests in Chapter 3.

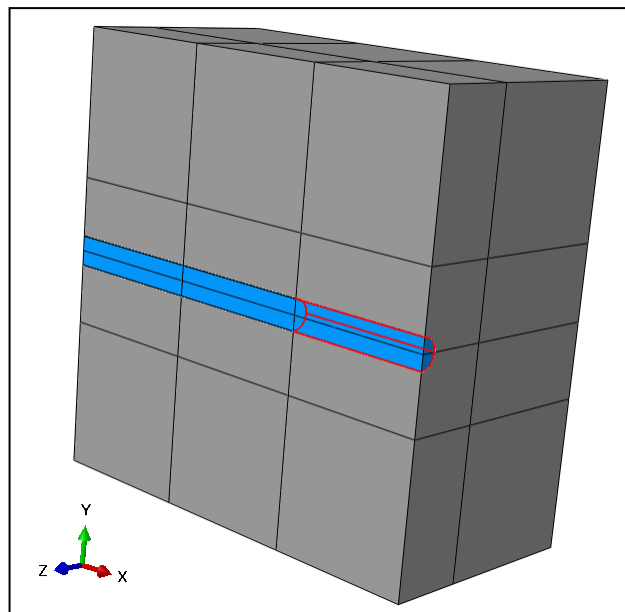


Figure 5.10. View of Abaqus model of pull-out specimen with surface which was assigned controlled displacement boundary condition outlined in red

Figure 5.11 shows the variation in pull-out force with increasing slip for simulated conditions alongside the equivalent experimental result for deformed reinforcement. Figure 5.12 is the equivalent plot for plain round reinforcement. An ambient temperature and a 290°C scenario have been included to provide a reasonable illustration of the bond modelling performance in both ambient and elevated temperature conditions. The small difference between the results is due to some limited discretisation error due to mesh sizing of elements and contact surfaces, and the increment size which produces fewer data points giving a less smooth curve than the experimental results.

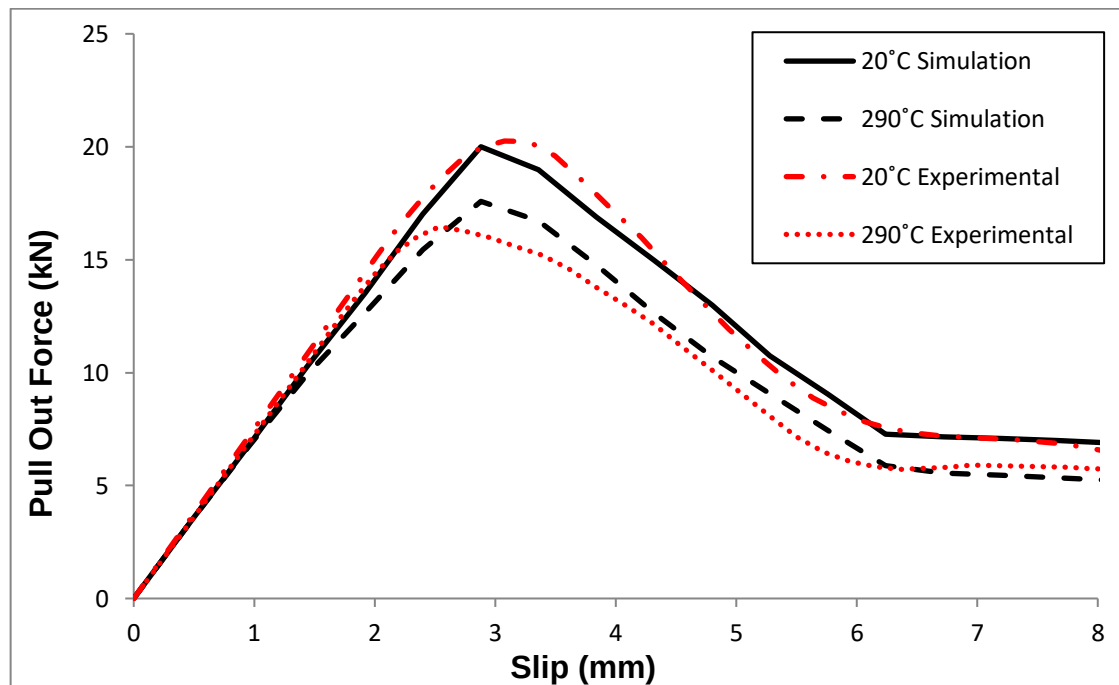


Figure 5.11. Comparison between pull-out force vs. slip of deformed bar from experimental results and Abaqus simulation

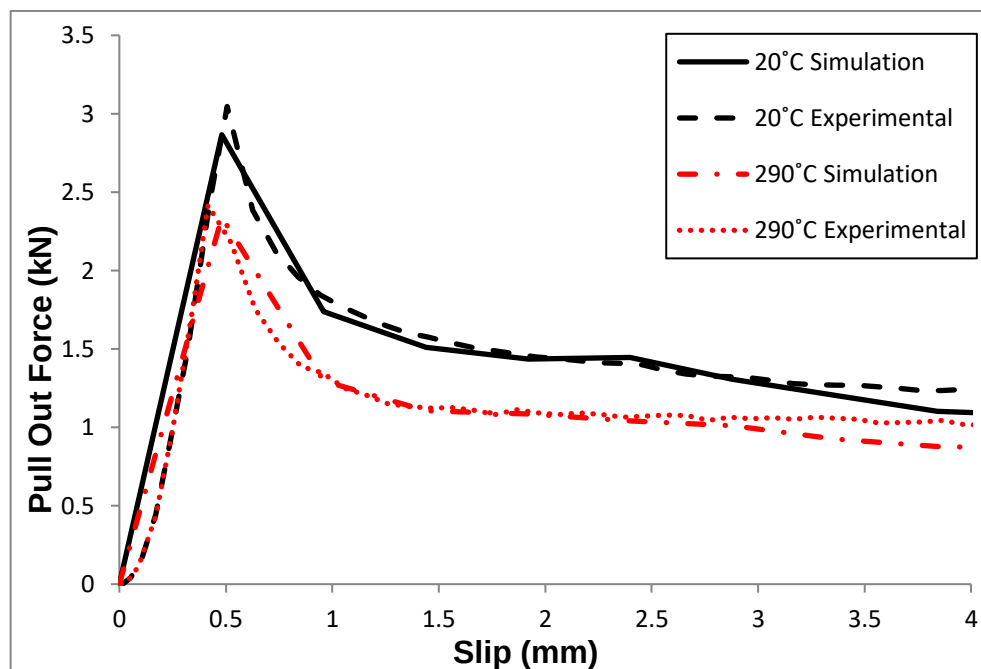


Figure 5.12. Comparison between pull-out force vs. slip of plain round bar from experimental results and Abaqus simulation

It is evident from figures 5.11 and 5.12 that the modelling approach is an effective way to model the bond stress-slip relationship. The values diverge to a small extent around the peak force for the deformed bar, but slip values this large are not encountered in any of the simulations so this will not be of any significant consequence to the research outputs.

5.6 Validation of modelling approach

A validation study has been conducted on the modelling approach and is explained in this section. The same modelling approach as explained previously has been used to develop an Abaqus model representing an ambient temperature experiment by Fayyad and Lees (2017). This particular experiment has been selected as the specimen has some similarity to the conceptual slab that was modelled, as described in Chapter 6, in that the specimen was of normal strength concrete with a single deformed reinforcing bar and was lightly reinforced.

The experiment which has been modelled was assigned the notation M30,D100,0.3 by Fayyad and Lees (2017). The experimental specimen had geometry as shown in figure 5.13. The reinforcing bar had a nominal diameter of 6mm and an average yield strength of 548MPa. The concrete cube compressive strength was 30MPa and 18mm concrete cover was provided to the reinforcing bar. The specimen also featured a 30mm deep pre-initiated crack at the mid-span, as shown in figure 5.13.

The specimen was tested to failure under a typical three-point bending arrangement at ambient temperature. Loading of the experimental specimens was by displacement control with the load applied at the mid-span. Further details of the experimental methodology can be found in (Fayyad and Lees, 2017). Figure 5.17 shows the load-deflection curve from the experiment by Fayyad and Lees (2017), alongside the output of the finite element study, discussed later. It was reported that the beam failed in flexure and that a crack propagated in the region of the highest applied moment.

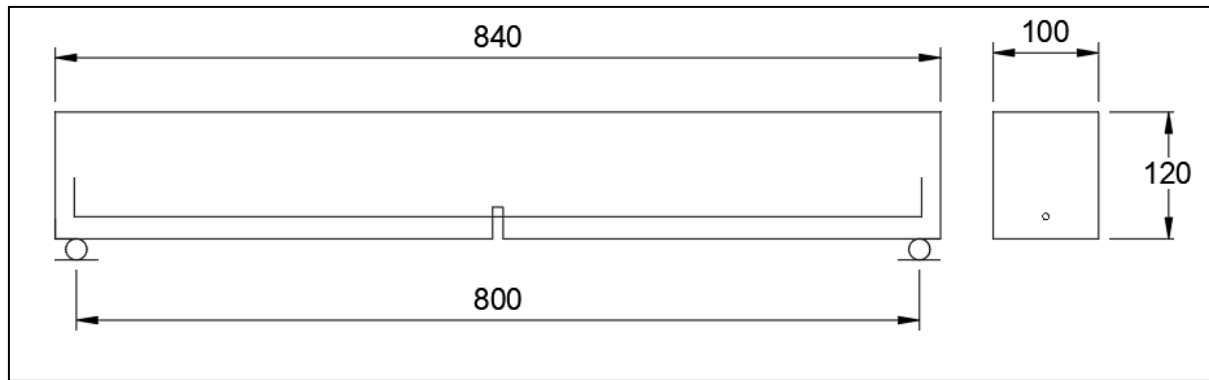


Figure 5.13. Experimental specimen from Fayyad and Lees (2017), with dimensions in millimetres

An Abaqus model was developed in the current research to represent the experimental specimen of Fayyad and Lees (2017). The modelling approach was the same as the slab model, described in chapter 6, except for a few details which are explained in the following paragraphs. As with the slab model, the modelling approach took advantage of symmetry by modelling only one quarter of the beam, as shown in figure 5.14, with appropriate boundary conditions.

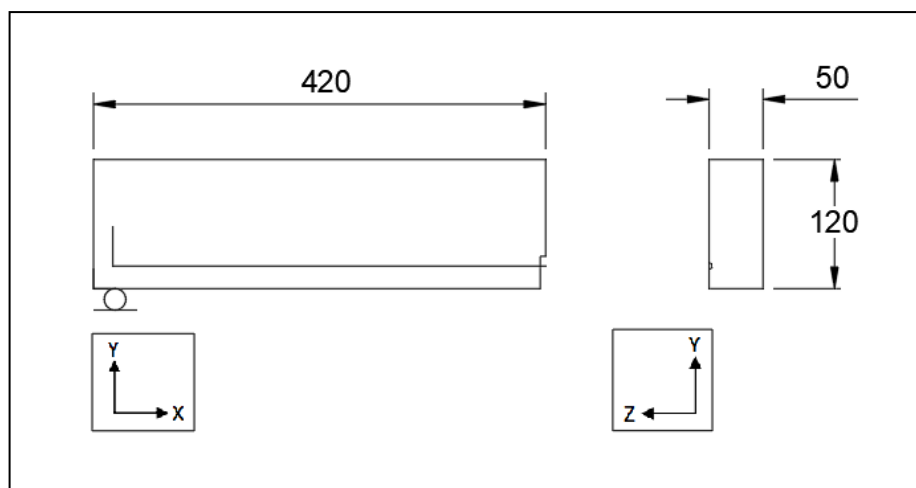


Figure 5.14. Specimen modelled in Abaqus, with dimensions in millimetres

A symmetric boundary condition was applied to the surface facing the Z direction and a fixed, or encastre, condition was applied to the surface facing the X direction.

The roller support was represented with a bearing plate which was set up with a controlled vertical displacement of 15mm, which ramped linearly throughout the analysis step, in the +Y direction, zero displacement in the Z direction and no other restraints. The bearing, therefore, acted similarly to a roller support in that it permitted rotation and movement in the X direction, but it also distributed the load acting on the concrete over a 10mm length. This approach meant that the support was displaced at a controlled rate while the mid-span remained static, but the overall effect on the beam was the same as in the experiment.

The reinforcement bond was represented in the same way as the models explained previously, with a cohesive surface interaction. The bond of the 6mm bar was assigned the same interaction properties as the 8mm bar in the slab models, which is considered reasonable because the properties are assigned to the surface on a per-unit-area basis. The anchorage provided at the ends of the reinforcing bar by the bar bend was represented by a tie constraint assigned over a 15mm length at the end of the bar, as outlined red in figure 5.15.

Materials were defined in the same manner as for the slab model, as described in section 5.3, but with some adjustment to align properties with those of the actual experiment. In the absence of an available stress-strain curve for the reinforcing bar from the study by Fayyad and Lees (2017), a stress-strain curve for a reinforcing bar of similar yield strength was taken from a study by Batson (2013), as shown in figure 5.16. The limiting strain for yield strength, ϵ_{au} , was taken from the curve as 0.107, matching figure 5.16. The concrete compressive and tensile behaviour inputs were derived in the same manner as for the slab model but recalculated based on the reported cube compressive strength of 30MPa.

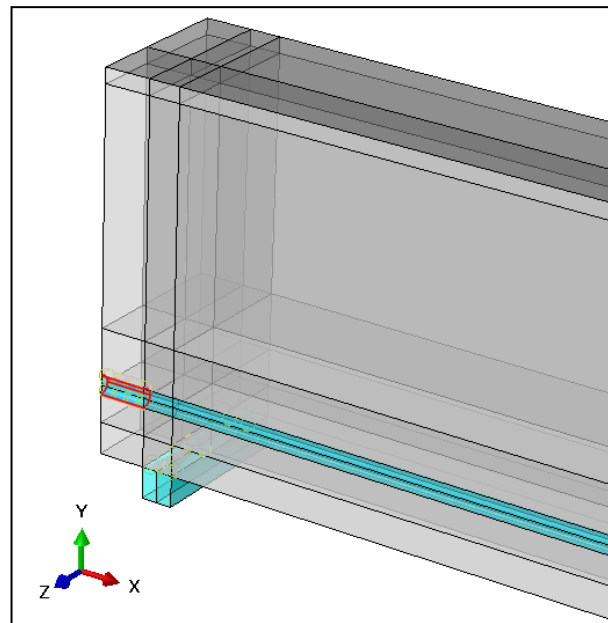


Figure 5.15. View of Abaqus model with the surface which was assigned a tie constraint outlined in red

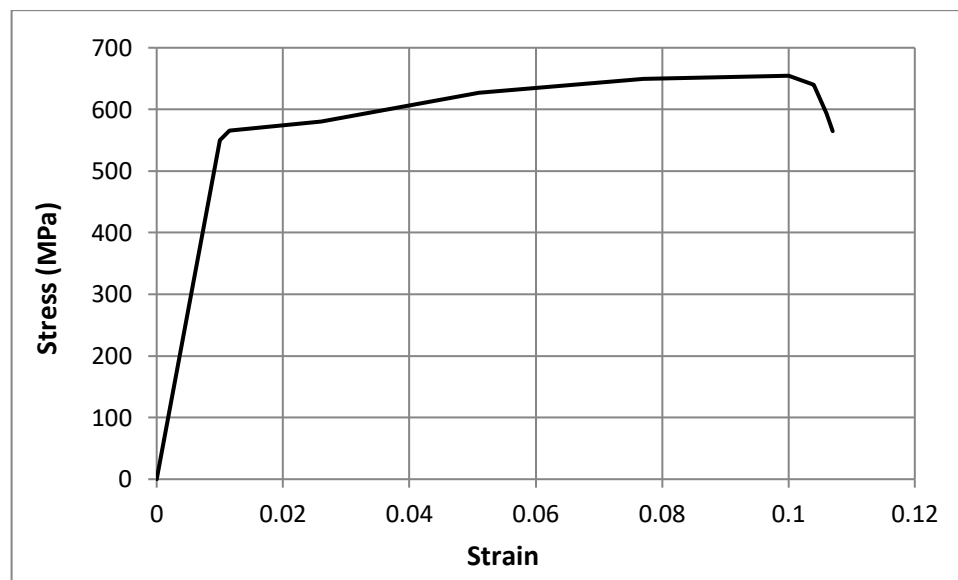


Figure 5.16. Stress-strain curve of reinforcing bars from a study by Batson (2013)

The load-deflection output of the simulation was taken from the centre-line of the bearing plate, where the controlled displacement was assigned. The load-deflection

result of the simulation is shown in figure 5.17, alongside the experimental result for comparison.

The simulation curve had a sudden drop in load and stiffness after the small initial deflection. This initial peak is less pronounced than that observed with the slab models due to the section being more balanced, in terms of flexural design, and the concrete being of a lower tensile strength. The experimental load-deflection curve also has an initial peak before a sudden drop in load, but at a larger deflection of around 1mm and it is less pronounced. It is expected that this was the result of a particularly large amount of crack development at this stage.

The initial stiffness of the simulation was greater than the experiment because the simulated specimen was uncracked and the tensile strength of the concrete is contributing to the flexural response at that stage. This is not observed in the experimental specimen as it will have initial imperfections and may have micro cracking. After the initial peak, the stiffness of the simulation response is similar to the experiment because of tensile damage which simulates cracking in the model and reduces the contribution of concrete in tension.

The remainder of the simulated load-deflection curve is quite similar to the experimental curve up to the point of failure of the experimental specimen. The applied load begins to plateau at around 3mm in both the experimental and simulated curves. The difference in the peak load is less than 3% and a final and rapid decline in load bearing capacity is onset at approximately 9mm. The fact that the simulation continues to show the reduction in load bearing capacity to a much lower level at around 10mm of deflection is not of significance to the interpretations which have been made in this research because the point of 'failure' can be

regarded as the point at which the bar reaches $\epsilon_{au,\theta}$. This is the point in the simulation where ductile damage is onset, beginning the rapid decline in load bearing capacity.

The validation study has demonstrated that the modelling approach produces simulations which are a reasonable representation of the behaviour of a real reinforced concrete beam when subjected to flexure and severely localised cracking. The level of accuracy demonstrated by the modelling approach is sufficient for the intended purpose and provides confidence in the reasonableness of the research findings.

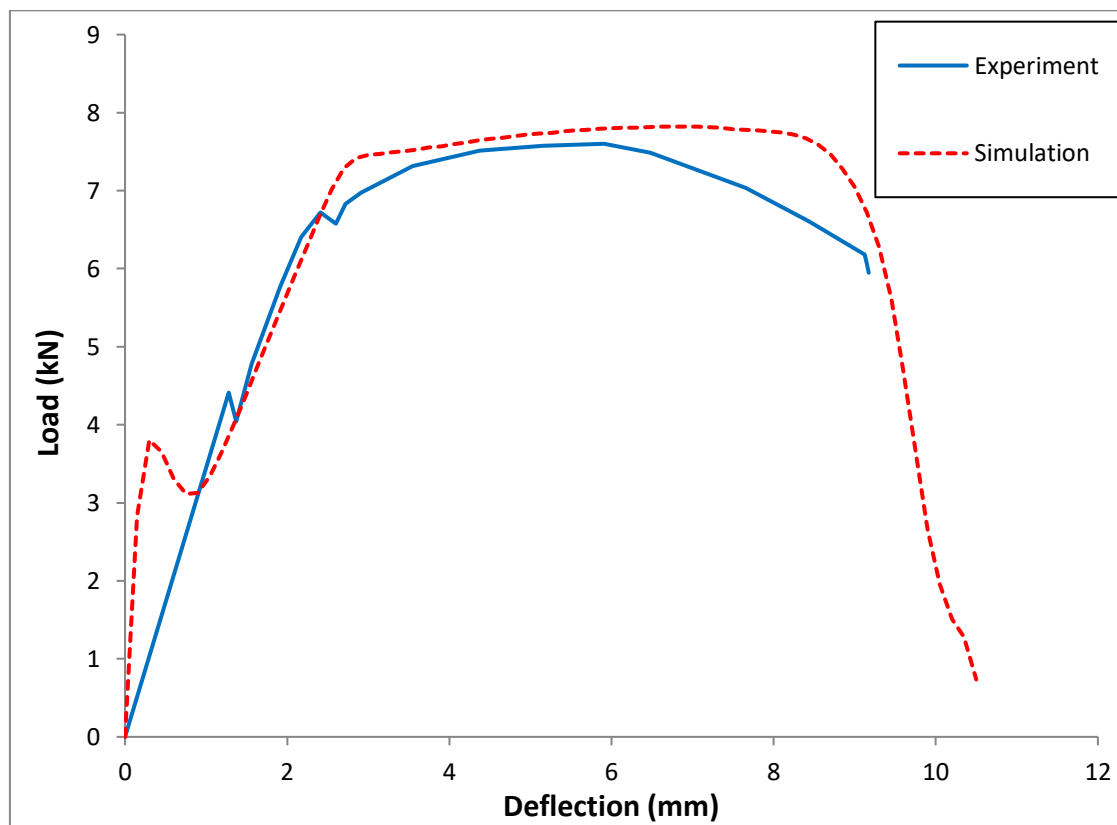


Figure 5.17. Comparison of load-deflection curves from Abaqus simulation and experimental results from Fayyad and Lees (2017)

6 Modelling rupture of slab perimeter reinforcement at elevated temperature

6.1 Introduction

This chapter describes the numerical model and simulations which were used to investigate the behaviour of reinforcement in composite and reinforced concrete slabs with continuity over perimeter supporting beams during the development of tensile membrane action at elevated temperatures. This model incorporates flexural crack development, bond slip and rupture of reinforcement bars as these are the most relevant behaviours to this research. The model uses the numerical modelling methodology which was described and validated in chapter 5. Section 6.2 describes the geometry, boundary conditions and loading of the numerical model. Section 6.3 describes the layout and size of mesh used in the model. The parameters which were investigated in this study are explained in section 6.4. Sections 6.5 and 6.6 describe the heat transfer analysis which was performed in order to provide a reasonable thermal gradient for elevated temperature simulations.

The slab specimen which has been modelled is a simplified representation of a zone within the hogging region of a composite or reinforced concrete slab with continuity over perimeter supporting beams. The modelled zone is indicatively shown as the shaded area in the top-down view of an area of slab shown in figure 6.1 below. The axes shown in figure 5.1 correspond to the axes of the Abaqus model assembly and this convention is consistent in the various figures presented. For clarity, figure 6.1 only shows the longitudinal, upper, reinforcing bars. Only these bars have been modelled as they are the bars which are of most interest in this case.

The model is of a slab with flat profile and composite action has not been modelled. This form of model was used in the interest of simplifying the model, whilst including the behaviours which are expected to be of most significance to the current research. In a scenario of tensile membrane action development during a fire, the steel decking will reach temperatures high enough that its stiffness and strength can be expected to become negligible, and the deck may also debond from the slab (Bailey et al., 2000; Jiang et al., 2022).

Composite slabs typically have a trapezoidal profile which will result in a different thermal distribution in the slab. This may result in higher temperatures at some reinforcing bars due to reduced concrete cover on the exposed lower surface. This is significant to the interpretation of the results, and to how the findings relate to composite slabs. This is a similar consideration to that of having different slab depths, which is investigated in this study. Additionally, simulations which were conducted at uniform elevated temperatures demonstrated the effect of having high temperatures in the region of the reinforcing bar. Although the model is of slab with flat profile, it is expected that the outputs will also be relevant, in context, to composite slabs with profiled decking as the localised behaviour of interest (strain localisation and potential rupture of reinforcement at hogging cracks) will be a common feature in both slab types.

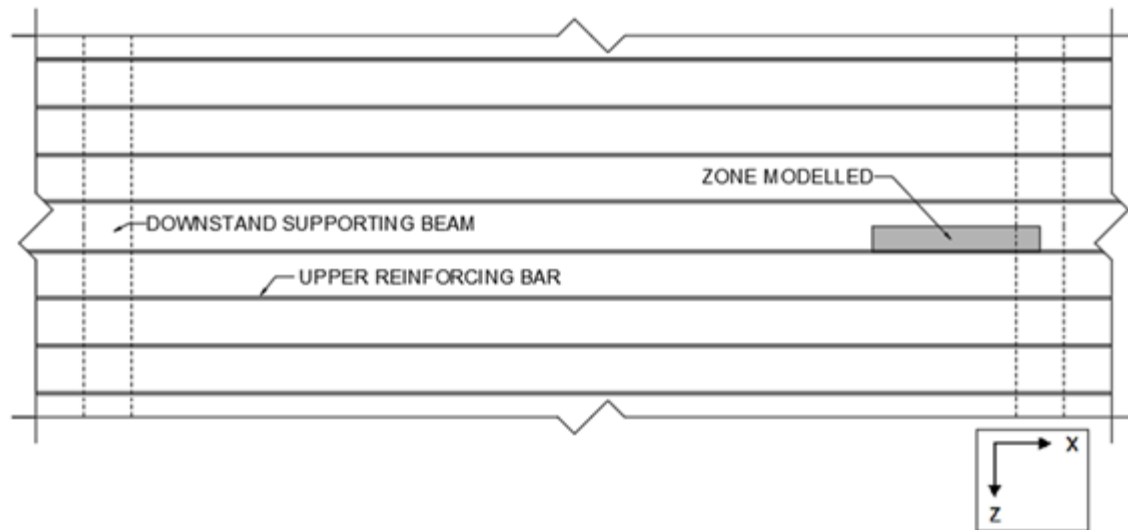


Figure 6.1. Plan view sketch showing the zone of a slab which has been modelled in Abaqus

Figure 6.2 is a three-dimensional view of the one of the Abaqus models, provided here as an example. The model shown in figure 6.2 is of a 200mm deep section of concrete slab with a 12mm diameter reinforcing bar, shown in blue. The green parts in figure 6.2 are bearing plates. The lower green bearing plate is representative of the vertical support provided by the outer edge of the upper flange of the downstand beam which supports the slab. The end of the beam facing the +X direction was assigned symmetry as a boundary condition to represent slab continuity across the supporting downstand beam. The boundary conditions and bearing plates are explained further in section 6.2.

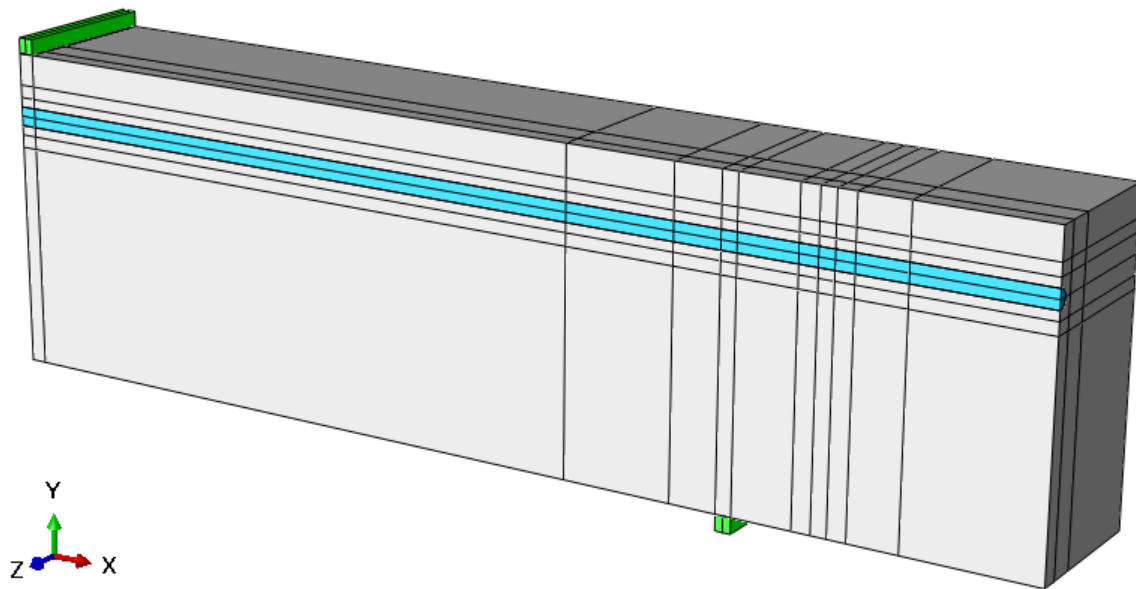


Figure 6.2. FE model of slab specimen with 200mm depth and 12mm diameter reinforcing bar

Modelling this individual part of a slab was computationally intensive, and increasing the size of the model would have reduced the number of simulations which could have reasonably been performed. Without modelling a specimen which represents an entire slab, the true thermal-stress state of a slab developing tensile membrane

action could not be correctly simulated. As the focus of this research is to investigate the very localised behaviour of the crack-spanning reinforcement, this limitation is of less significance. Crucially to this research, the bond interaction behaviour which is used in the model is sourced from values fitted to experimental data which is already substantially affected by thermal strain, as discussed in chapter 3. If thermal strain was present in the FE model then the effects of thermal strain on the bond response would be accounted for twice, changing the bond response in an unrepresentative way. For these reasons, thermal strain is considered exclusively through the bond definition, and an uncoupled heat transfer analysis approach is adequate and preferable to a coupled approach.

As this is a comparative study of reinforcement rupture under different conditions, failure is considered to occur at the point at which the ultimate strain, $\varepsilon_{ae,\theta}$, is reached in the reinforcing bar because this is the stage at which rupture would occur. The analysis of the results, presented in chapter 7, is conducted from the context of this definition of failure.

The overall modelling approach was validated at ambient temperature, as described in section 5.6. The thermal gradient within the slab is considered to be reasonable on the basis that the temperatures are in reasonable agreement with those provided in Eurocode 4 Part 1-2 as an approximation of temperatures at different depths in a slab under the ISO standard fire. The effect of temperature on the bond interaction is complex and includes thermal strain effects, but this has been sourced from elevated temperature experimental data and the bond modelling approach has been validated at elevated temperature, as described in section 5.5. The damage modelling of concrete follows recommendations of a calibration study which was performed by

Wahid et al. (2019), where concrete damage plasticity was calibrated to elevated temperature experimental data. This gives confidence that the effect of elevated temperature on the most significant behaviours, of bond interaction and concrete damage, will be represented reasonably well. The overall modelling approach, however, could not be validated at elevated temperature due to the lack of suitable real or experimental data for validation. The lack of a more extensive elevated temperature validation is a limitation of the current research.

6.2 Model geometry, boundary conditions and loading

This section describes the geometry of the main FE model. This geometry differs from the validation models described in sections 5.5 and 5.6, although the overall modelling methodology was the same. The geometry of the modelled specimens is shown in figure 6.3. The diameter of the reinforcing bar varied between scenarios with values of 8, 10 and 12mm. The slab depth varied between scenarios and was either 150mm or 200mm. The dimensional values which were varied between scenarios are marked with an asterisk, and all other dimensional values were consistent for all scenarios.

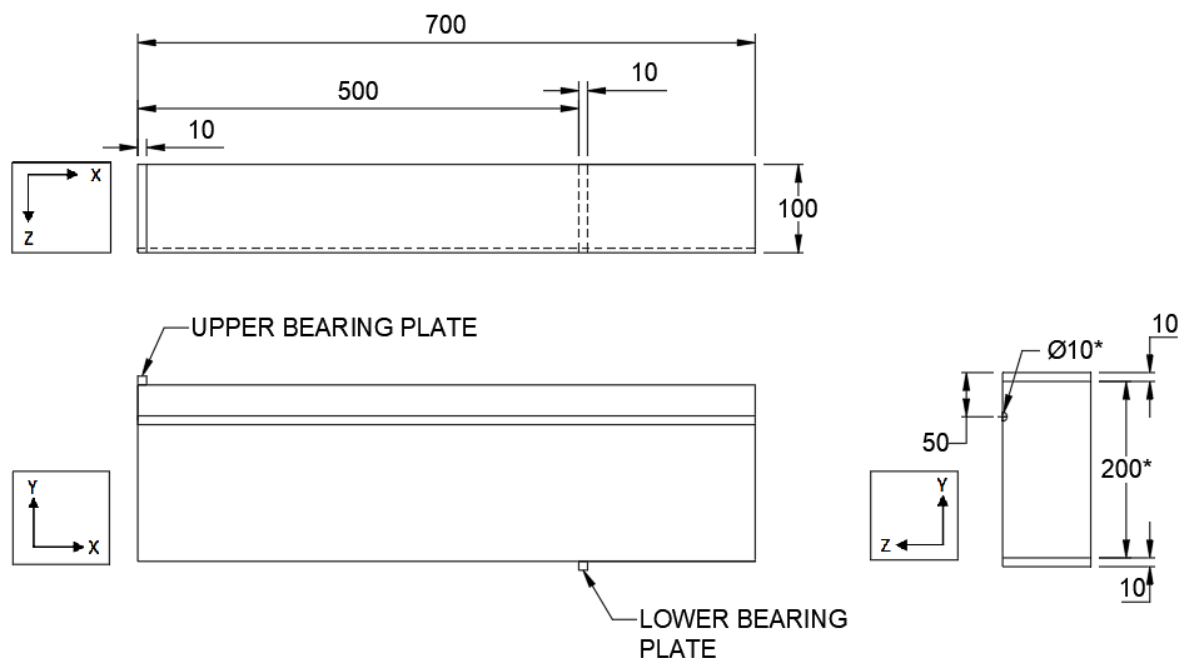


Figure 6.3. Drawing of the modelled specimen, with dimensions in millimetres

The specimen width was chosen as the reinforcement mesh in the UK has bars at 200mm spacing, and the specimen represents a 200mm wide slice of the slab with a single bar. Only 100mm width is modelled because the model takes advantage of symmetry. Provided the model has enough cover in all directions to provide

reasonable confinement, the model width is not expected to be a very significant dimension from the perspective of this study. The specimen length of 700mm was chosen as it is a reasonable length to induce flexure from a controlled vertical displacement at the upper bearing plate, but not unnecessarily long as a larger model would have increased computational time.

The concrete slab and reinforcing bar were assigned symmetry boundary conditions to the surfaces which face the positive X and Z directions. These were exclusively the surfaces highlighted red in figure 6.4. The upper and lower bearing plates reduced the concentration of stress in the slab from the boundary conditions.

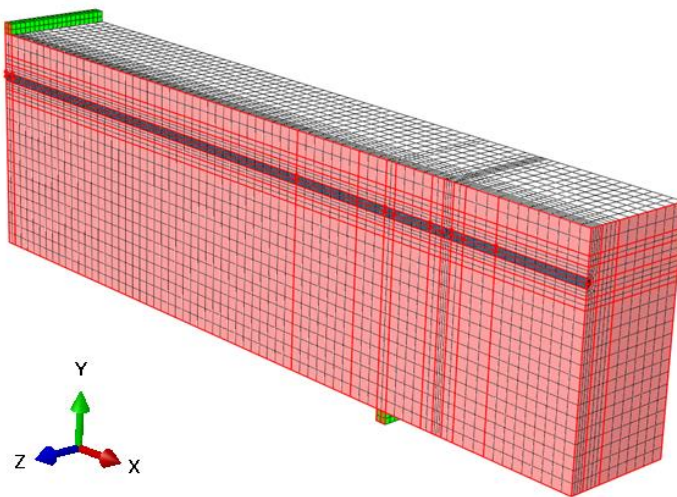


Figure 6.4. View of model with the surfaces which were assigned symmetry boundary conditions highlighted in red

As tensile membrane action develops, deflection of the slab increases and there is a rotation of the slab around the boundaries. This rotation is facilitated by the development of large hogging cracks. A controlled displacement was applied in the model to represent this deflection and to induce bending and cracking in the slab specimen. Although the stress distribution in the slab is not produced in exactly the

same way as a full slab in tensile membrane action, the focus of this research is on the very localised behaviour of the crack-spanning reinforcement and not the wider stress distributions and, therefore, the use of controlled displacements in this way is considered reasonable. Additionally, the effect of increased loading and stress in the reinforcing bars was considered, as explained later in this section.

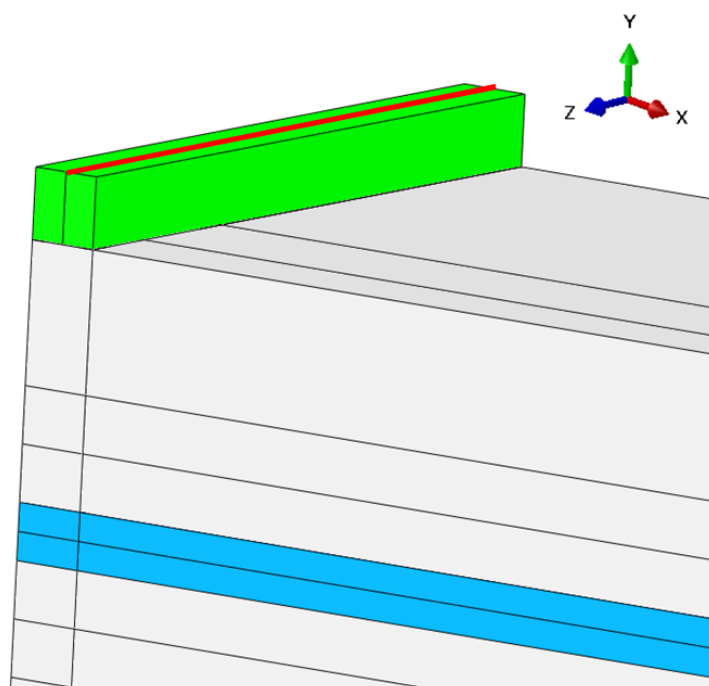


Figure 6.5. View showing the upper bearing plate of the FE model, with the edge which was assigned the controlled displacement marked in red, and the bearing plate shown in green

Displacements were assigned to bearing plates in order to avoid excessive concentrations of compressive stress at those locations, as this was not a focus of the investigation. The bearing plates were assigned the same material properties as the steel, except with no ductile damage function. The material properties are described in section 5.3. The top of the upper bearing plate was assigned a downward ($-Y$) controlled displacement on the centreline which ramped linearly

through the analysis step. On the bearing plate, all degrees of freedom other than vertical translation were unrestrained. The lower surface of the upper bearing was constrained to the opposing concrete surface with a tie constraint, with the upper bearing being the master surface of the pair. The lower bearing plate was set up in a similar manner but was assigned a zero displacement boundary condition in the Y direction on the centreline of its lower surface and the tie constraint was on its top surface.

The geometry and loading conditions are intended to represent a portion of a slab, but with some simplifications. The symmetry in the +X direction represents the continuity of the slab across the vertical support, at one edge of the slab bay. The symmetry in the +Y direction is simply to allow only half of the portion of the slab to be modelled in the interest of reducing computational time. The lower bearing plate is intended to represent the outer edge of the upper flange of a supporting downstand beam, as it is assumed that the slab would bear upon this edge due to the flexural hogging. In reality, this edge would have a finite stiffness against vertical translation and may displace slightly under the applied load, but it is assumed that this effect would be small and would not significantly influence the development of the flexural crack which is of interest. The position of the upper bearing plate and controlled displacement is not intended to represent a real point of loading of the slab; it is a simplification of the model which allowed flexural hogging to be induced. The upper bearing plate was placed at the -X end of the specimen in order to avoid localised stresses from the bearing plate interfering with the region where the cracking develops.

In the real situation of interest, the reinforcing bar is in tension due to flexural hogging and due to the membrane forces. The flexural hogging is induced in the model by the controlled displacement, but the membrane forces are not simulated in the model. As the intention of the model is to investigate the effect of various parameters on the rupture of the crack-spanning behaviour of reinforcement as a comparative study, rather than to predict the point of failure of a particular slab, it was considered reasonable to omit the tensile membrane forces, provided that this approach was consistent and provided that it does not meaningfully change the localised behaviour of the crack-spanning reinforcement. Omission of these forces was preferred because they are unknown and would be complex to simulate, and a simulation was conducted to confirm the appropriateness of this assumption.

One simulation was conducted with an additional tensile force on the bar in order to determine what effect this would have on the localised behaviour of the crack-spanning reinforcement. Although the membrane action is not simulated in the model, it is still possible to determine whether significantly increased tensile force in the bar will meaningfully change the localised behaviour of the crack-spanning reinforcement. For this purpose, the additional force does not need to match that of actual membrane forces, provided that it is at least comparably onerous, in magnitude and application, to realistic membrane forces in order to demonstrate whether it is reasonable to omit the membrane forces from analysis.

The additional force was applied in the simulation numbered 6 in table 6.2. In this simulation, a pressure of 200MPa was applied to the -X end surface of the reinforcing bar in the -X direction. The magnitude of 200MPa on the bar was chosen as it is a significant proportion of 500MPa, which is the yield strength of the bar and it is therefore large enough to represent a moderate additional loading of the

specimen. For clarity, there is nothing special about the value of 200MPa in this instance, and a value such as 300MPa would also have been reasonable. It is expected that if an additional force of this magnitude does not meaningfully change the behaviour of the crack-spanning reinforcement, in terms of the bond slip and the distribution of axial strain at bar rupture, then the actual membrane forces would be unlikely to do so either because the maximum possible membrane forces are limited in magnitude by the capacity of the bar.

The additional force which was applied increased linearly throughout a step which occurred after the heat transfer step, but before the step in which the controlled displacement was applied. This is a simplification which is not expected, or intended, to represent the true development of membrane forces in a real fire scenario, where membrane forces may vary as displacement increases. This is considered to be conservative, however, because the applied force is at maximum magnitude as displacements increased. The force remaining at maximum magnitude is expected to have at least as significant an effect on crack-spanning reinforcement as a condition where the forces vary gradually and are either lower than or equal to the maximum magnitude at any point in time.

6.3 Discretisation

The mesh arrangement of the slab FE model is shown in figures 6.6 and 6.7. The model had varied mesh spacing, with finer mesh in the region around the vicinity of the expected crack and bar rupture. This region had been identified from various trial runs during model development. The finer mesh was 2.5mm and the coarser mesh was 10mm with a stepped transition between mesh sizes. Due to the round shape of the reinforcing bar, some elements were finer than these typical sizes. The appropriateness of the mesh size was established by a sensitivity study, as explained in section 7.2.

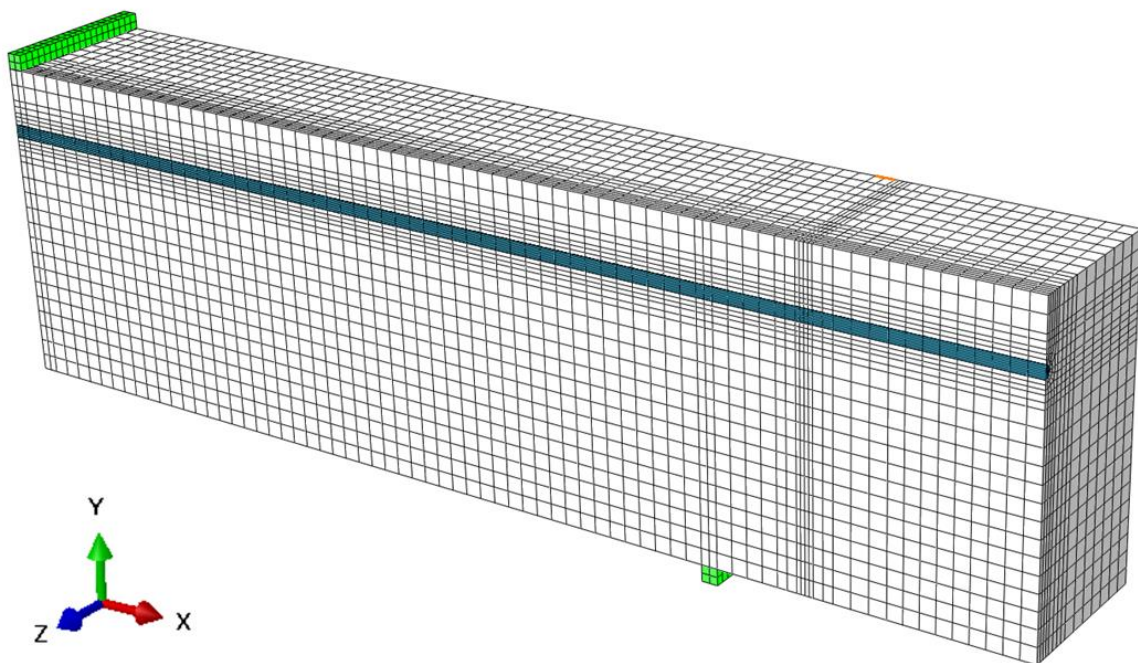


Figure 6.6. View of model assembly with 8mm diameter bar, showing the mesh arrangement, with the bar shown in blue and bearing plates in green

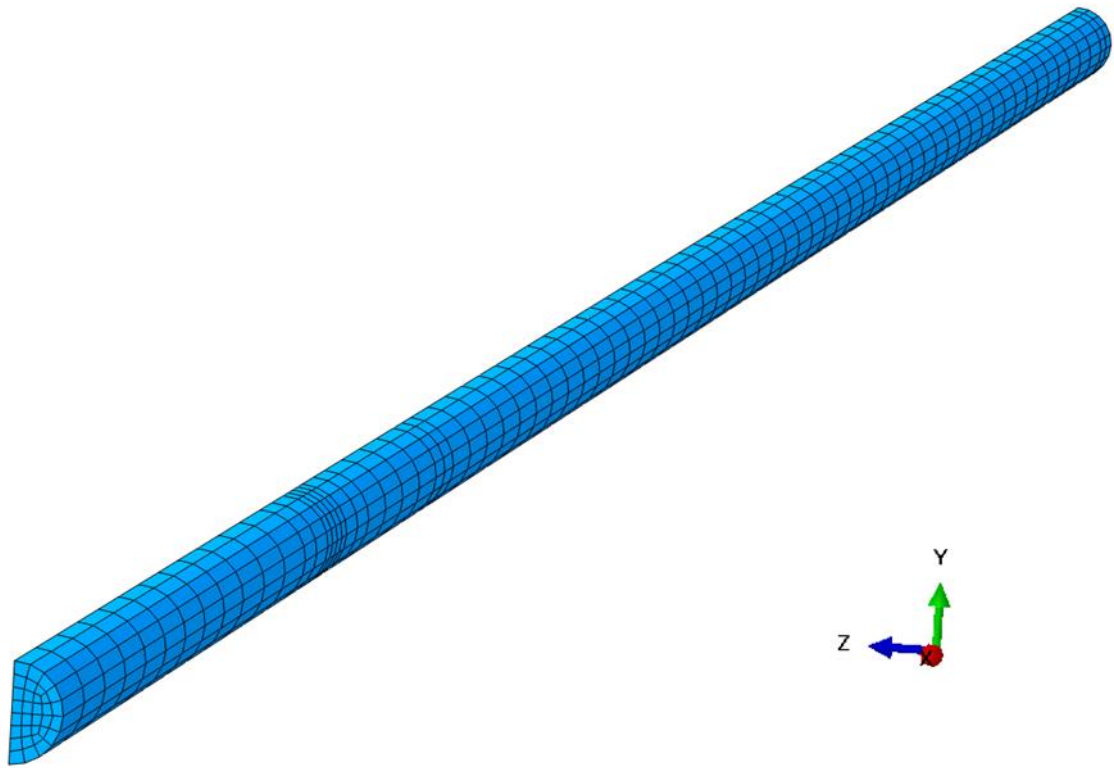


Figure 6.7. View showing mesh arrangement on reinforcing bar

6.4 Parameters investigated

The behaviour of the crack-spanning reinforcement will depend on various parameters including bar diameter, bar type, slab depth and temperature. Table 6.1 shows the different parameters which have been considered in simulations using the FE model. The combinations of each of these parameters which were considered in simulations are listed in table 6.2.

Parameter	Values considered
Bar diameter	8mm
	10mm
	12mm
Bar surface/bond type	Plain round bar
	Deformed bar
	Perfectly bonded bar (with no bond slip)
Slab depth	200mm
	150mm
Temperature	20°C
	200°C
	300°C
Fire duration	0 minutes (20°C)
	60 minutes
	120 minutes
Loading	None applied
	Additional load applied*
Ductility	Increased ductility**
	Reduced ductility**
Steel yield strength	500 N/mm ²
	250 N/mm ²
* Explained in section 6.2, ** Explained in section 6.4.	

Table 6.1. Parameters and values considered in FE model simulations

Differences in the ratio of cross sectional area to bond surface area of different bar diameters will change the ratio between axial stress in the bar and bond stress and, therefore, it is reasonable to expect that this would have some effect on the relationship between rupture of the bar and bond slip. It is important to understand

this effect as different bar diameters may make a slab more or less prone to rupture of the crack-spanning reinforcement in the scenario being considered. Bar diameters of 8mm, 10mm and 12mm were considered as these are the diameters which were investigated in the pull-out tests and, therefore, there is compatible experimental data on their stress-slip response.

As described in chapter 2 and investigated in chapter 3, plain and deformed bars have different bond interactions with concrete. It is not known from previous research, however, how deformed bars would behave in this scenario and if their bond characteristics would render them more prone to rupture. Hence, the relative performance of plain and round reinforcement under different conditions has been investigated with the FE model. The assumption of perfect bond between reinforcement and concrete is a common assumption in design models. The reasonableness of this design assumption in the scenario being considered has been investigated with the FE model.

The tensile membrane action design methods are based on the observations of the Cardington tests, where a 130mm deep slab was considered (Bailey, 1999). Modern composite slabs are often considerably deeper than this. It is not fully understood how increased slab depth will affect the ability of a slab to develop tensile membrane action without rupture of the reinforcement around the slab boundaries. The hogging reinforcement will be further from the point of crack rotation in a full depth hogging crack within a deeper slab than a shallower slab. It is reasonable, therefore, to expect that this will affect the ability of the reinforcement to withstand cracking and rotation of the slab near the slab boundaries as tensile membrane action develops. Depths of 150mm and 200mm have been considered in simulations as these are reasonable slab depths which are within the normal range in construction.

Temperature changes the material properties of concrete and steel, as described in section 2.6. Temperature also affects the bond response of plain and deformed reinforcing bars, as discussed in section 3.3. It is reasonable to expect, therefore, that temperature will affect the behaviour of crack-spanning reinforcement in the fire scenario being considered, but these effects are not fully understood from previous research. In order to understand the effects of temperature, and temperature gradients within a slab, both uniform temperatures and thermal gradients due to heat transfer over a fire duration have been considered. Temperatures of 20°C, 200°C and 300°C were considered as uniform temperatures because these temperatures were found to affect the bond of plain and deformed bars differently in chapter 3, and it is more probable that reinforcement would reach these temperatures than higher temperatures. Fire durations of 60 minutes and 120 minutes were also considered as these are typical values of building design requirements (BSI, 2015).

In tensile membrane action, the hogging reinforcement carries tensile forces from the tensile membrane. These forces are not present in most of the simulations. In one simulation, an additional force was applied directly to the reinforcement to represent these tensile forces. This was conducted to determine whether omitting this force significantly affected the crack-spanning behaviour of the reinforcement. Details of the load and its application are described in section 6.2.

In the UK, plain round reinforcement typically has a specified yield strength of 250N/mm² and deformed reinforcement typically has a specified yield strength of 500N/mm². In one simulation a plain round bar was simulated with 500 N/mm² yield strength as this allowed the effect of surface type to be investigated in isolation, without differences in materials, and it was helpful in interpreting the results.

These materials also differ in terms of ductility. Clearly, ductility and material strength will affect the rupture behaviour of crack-spanning reinforcement. Different values of ductility have been considered for both types of reinforcement bars in order to understand the effect of these parameters. The ductility is varied between simulations by changing the values of two strains. These are the limiting strain for yield strength in the fire situation, $\varepsilon_{au,\theta}$, and the ultimate strain in the fire situation, $\varepsilon_{ae,\theta}$.

The Eurocode strain values were preferred as the base condition for simulations as they are temperature dependent and they are clearly defined. This allows a consistent approach to be taken across simulations at all temperatures. However, these Eurocode curves consider ductility to be equal for different steel grades, which is not realistic. The ductility of typical grade 500B deformed reinforcing bars and 250 grade plain round bars is quite different. In order to evaluate the significance of ductility as a parameter, the simulations which were numbered 7 and 13 use more realistic values of $\varepsilon_{au,\theta}$, and $\varepsilon_{ae,\theta}$, based on a study by Du *et al.* (2015).

Strain values of $\varepsilon_{au,\theta}$ and $\varepsilon_{ae,\theta}$ of 0.18 and 0.23 respectively have been used for the plain bar in the simulation referred to as having 'increased ductility', which is simulation 13 in table 6.2. The values of $\varepsilon_{au,\theta}$ and $\varepsilon_{ae,\theta}$ have been taken as 0.08 and 0.14 respectively for the deformed bar in the scenario referred to as having 'reduced ductility', which is simulation 7 in table 6.2. These values of $\varepsilon_{au,\theta}$ and $\varepsilon_{ae,\theta}$ are based on a study by Du *et al.* (2015) and are based on experimental data for similar reinforcing bars. These strain values are, therefore, expected to be more realistic.

Test	Bar Diameter (mm)	Bar surface/ bond type	Slab depth (mm)	Temperature or Fire Duration	Loading	Ductility definitions	Steel yield strength (N/mm ²)		
1	8	Deformed	200	20°C	No additional	$\varepsilon_{au,\theta} = 0.15$ $\varepsilon_{ae,\theta} = 0.2$	500		
2				60 minute					
3				120 minute					
4				200°C					
5				300°C					
6				120 minute	Additional				
7				No additional		$\varepsilon_{au,\theta} = 0.08$ $\varepsilon_{ae,\theta} = 0.14$			
8								$\varepsilon_{au,\theta} = 0.15$ $\varepsilon_{ae,\theta} = 0.2$	
9									250
10									
11		120 minute							
12			300°C						
13				120 minute					
14					$\varepsilon_{au,\theta} = 0.18$ $\varepsilon_{ae,\theta} = 0.23$				
15						$\varepsilon_{au,\theta} = 0.15$ $\varepsilon_{ae,\theta} = 0.2$			
16		Perfect							
17	10		Deformed				20°C		
18				120 minute					
19					120 minute				
20						20°C			
20		120 minute							

Table 6.2. Combinations of parameters considered in simulations

6.5 Heat transfer

The analysis was performed by firstly conducting the pure heat transfer analysis on a modelled specimen which was geometrically identical to the structural model. The temperature solution was then imported into the stress analysis via a predefined field input. This is only relevant for the simulations which were conducted after a fire duration, as identified in table 6.2. Temperature in the other simulations was assigned directly using a predefined field.

Conductivity has been taken as isotropic for the materials in this study and therefore only one conductivity value is specified at each temperature for each material. The temperature dependent thermal conductivity of steel has been taken in this study as per the recommended values in Eurocode 4 Part 1-2 (BSI, 2005b). The temperature dependent thermal conductivity of concrete has also been taken in this study as per the recommended values in Eurocode 2 Part 1-2 (BSI, 2019).

In this study, the effects of convection are represented in Abaqus by a surface film condition interaction. This type of interaction includes definitions of a sink temperature and a film coefficient for convection. The sink temperature for the unexposed surface was taken as 20°C. The exposed surface was assigned a time dependent sink temperature equal to the standard fire temperature-time curve from Eurocode 1 Part 1-2 (BSI, 2002).

The value of the convection coefficient is generally found to lie between 10 and 30 W/m² K (SFPE, 2004). Values of 12 W/m² K and 20 W/m² K were taken in this study for the unexposed and exposed surfaces respectively. In hindsight, it may have been more advisable to have taken the values of 4 W/m²K and 25 W/m²K for the

unexposed and exposed surfaces respectively, as recommended in Eurocode 1 Part 1-2 (BSI, 2002).

The values taken for the convection coefficient in this study were, however, within the realistic range (SFPE, 2004). After re-running the heat transfer analysis with the Eurocode recommended values, the temperatures were only around 1°C different from the outputs of the original heat transfer analysis used in this study. This difference is negligible within the context of this study and the original heat transfer outputs are considered sufficient. In typical fires, heat transfer is generally not sensitive to the convection coefficient because thermal radiation dominates heat transfer at these temperature ranges (Buchanan, 2017).

An emissivity value of 0.7 has been taken for the concrete surface, as recommended by Eurocode 2 Part 1-2 (BSI, 2019). An emissivity value is not required for the steel as the surfaces of the steel are not exposed. The emissivity of the fire has been taken as 1.0 as per the recommendation of Eurocode 2 Part 1-2 (BSI, 2019). The configuration factor, which is a measure of the extent to which the emitting surface is 'seen' by the receiving factor, and has been taken as 1.0, as recommended by Eurocode 2 Part 1-2 in the absence of specific data. The effective radiation temperature of the fire environment has been taken as the standard fire temperature-time curve from Eurocode 1 Part 1-2 (BSI, 2002). The surface radiation properties were assigned using a surface radiation type interaction in Abaqus.

The specific heat capacity of concrete and steel have been taken as the temperature dependent values from Eurocode 2 Part 1-2 (BSI, 2019) and Eurocode 4 Part 1-2 (BSI, 2005b) respectively. As per the recommendation of Eurocode 4 Part 1-2 (BSI, 2005b), the density of steel has been taken as a temperature independent value of

7850kg/m³. Temperature dependent values of mass density for concrete have been taken as those given in Eurocode 2 Part 1-2 (BSI, 2019)

The geometry of the heat transfer model was identical to the structural model. Mesh was also identical to the structural model. The entirety of the heat transfer model was assigned Abaqus element type DC3D8, which is a basic 8-node linear heat transfer brick. A time period of 7200 seconds was defined within the heat transfer step.

'Surface film condition' type interactions were assigned to the unexposed top surface and the exposed surface as highlighted red in figure 6.8 below. 'Surface radiation' type interactions were assigned to the unexposed and exposed surfaces in the same manner. These surfaces are the only boundaries across which heat flux enters or exits the modelled specimen during the heat transfer analysis. A tie constraint was assigned to the steel and concrete faces along the bond interface, with the concrete assigned as the master surface of the pair, to facilitate heat transfer between the bonded materials.

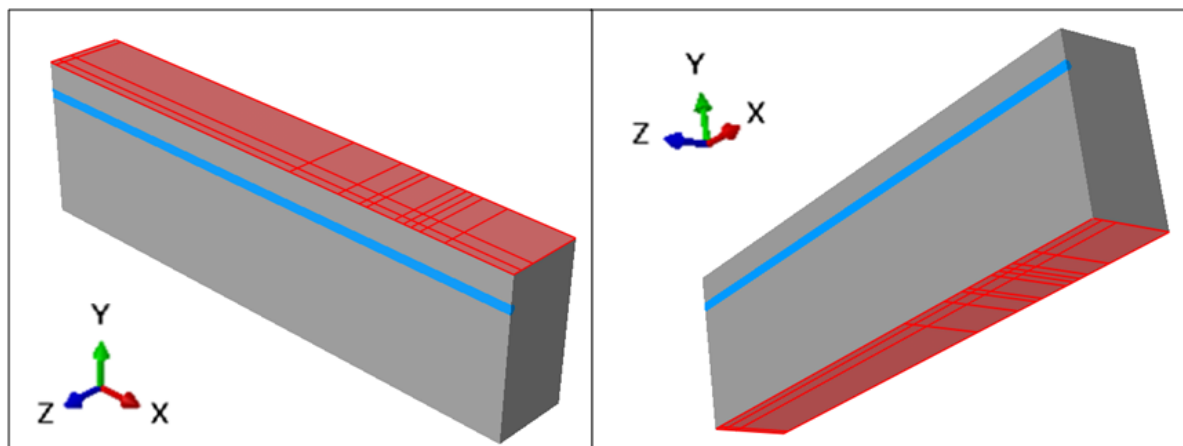


Figure 6.8. Unexposed surface (highlighted red on left) and exposed surface (highlighted red on right).

6.6 Results, analysis and discussion of thermal analysis

This section provides the results, analysis and discussion of the thermal analysis of the slab. Chapter 7 includes the results, analysis and discussion of the various structural simulations which were conducted using the FE model. Section 7.2 presents a mesh sensitivity study.

The heat transfer outputs serve as an input to the structural models via a predefined field reference. The results of the heat transfer analysis are presented in the form of contour plots for different bar diameters and slab depths in figures 6.9, 6.12, 6.14 and 6.16. Figures 6.10, 6.13, 6.15 and 6.17 show the variation in temperature with distance from the bottom of the slab for the 200mm deep slab with 8mm diameter bar at 120 minutes fire duration at the location shown in figure 6.11.

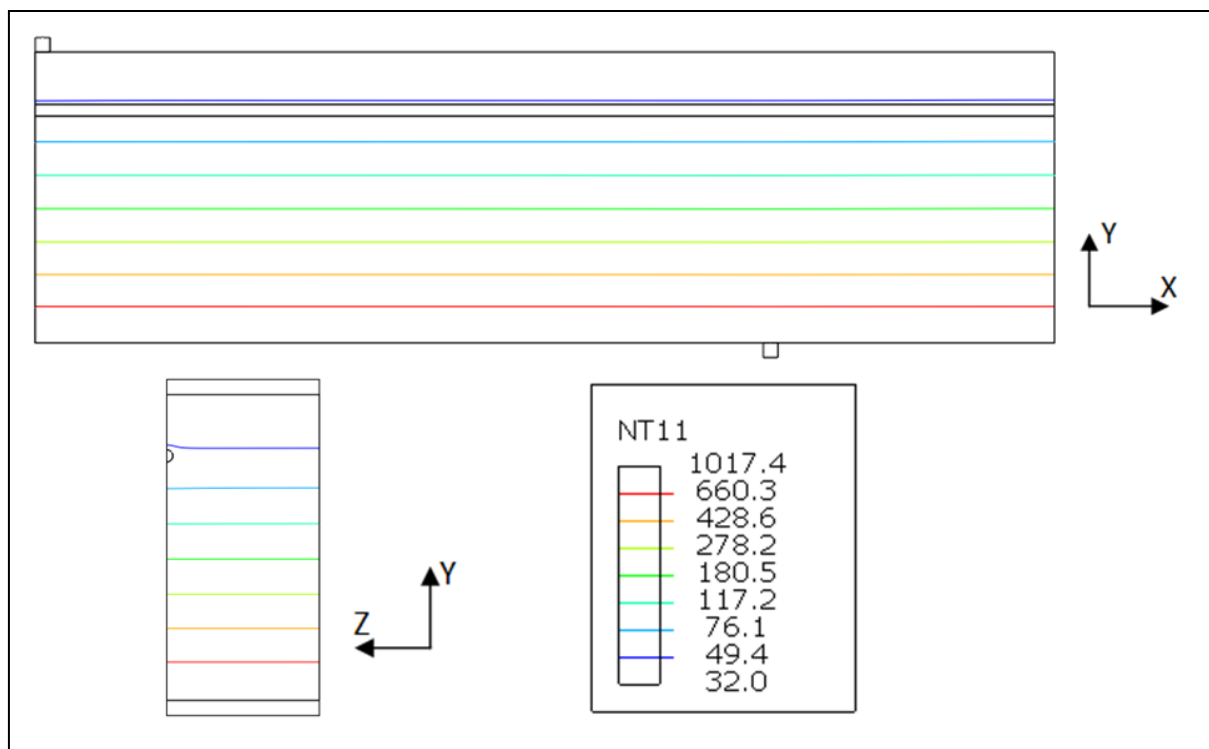


Figure 6.9: Temperature contour plot for 200mm deep slab with 8mm diameter bar at 120 minutes fire duration

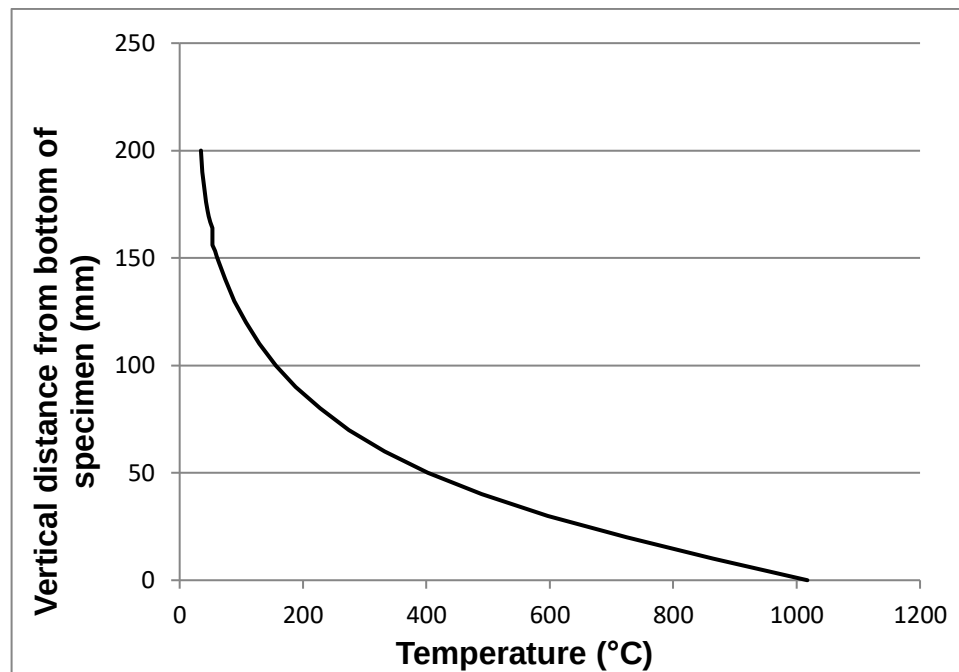


Figure 6.10. Temperature variation with distance from the bottom of the slab for the 200mm deep slab with 8mm diameter bar at 120 minutes fire duration at the location shown in figure 6.11.

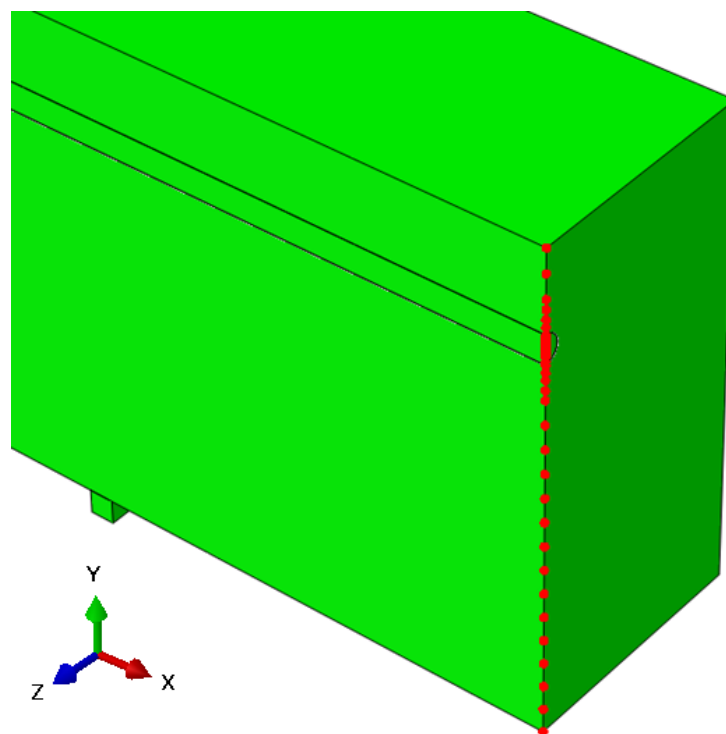


Figure 6.11. View of model showing the position of the nodes (highlighted red) used to produce figures 6.10, 6.13, 6.15 and 6.17

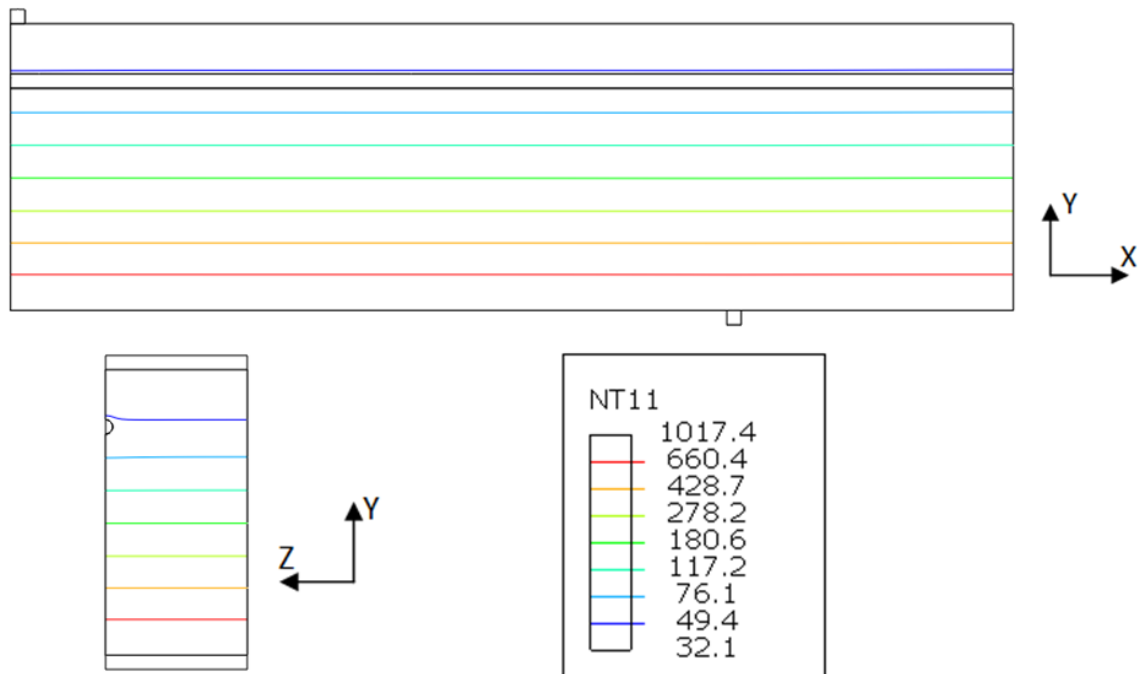


Figure 6.12: Temperature contour plot for 200mm deep slab with 10mm diameter bar at 120 minutes fire duration, values in °C.

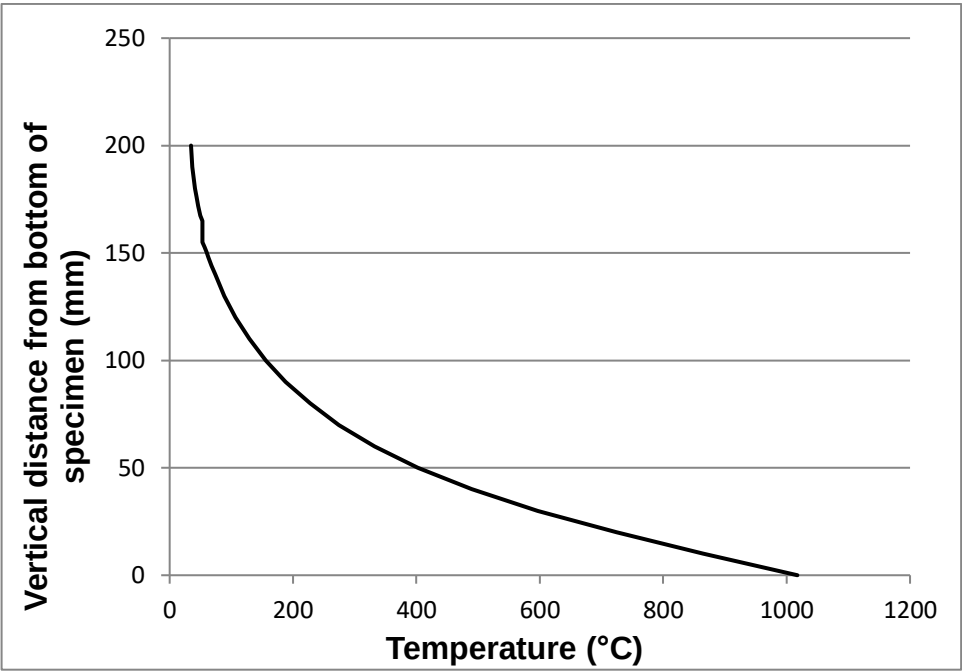


Figure 6.13. Temperature variation with distance from the bottom of the slab for the 200mm deep slab with 10mm diameter bar at 120 minutes fire duration at the location shown in figure 6.11.

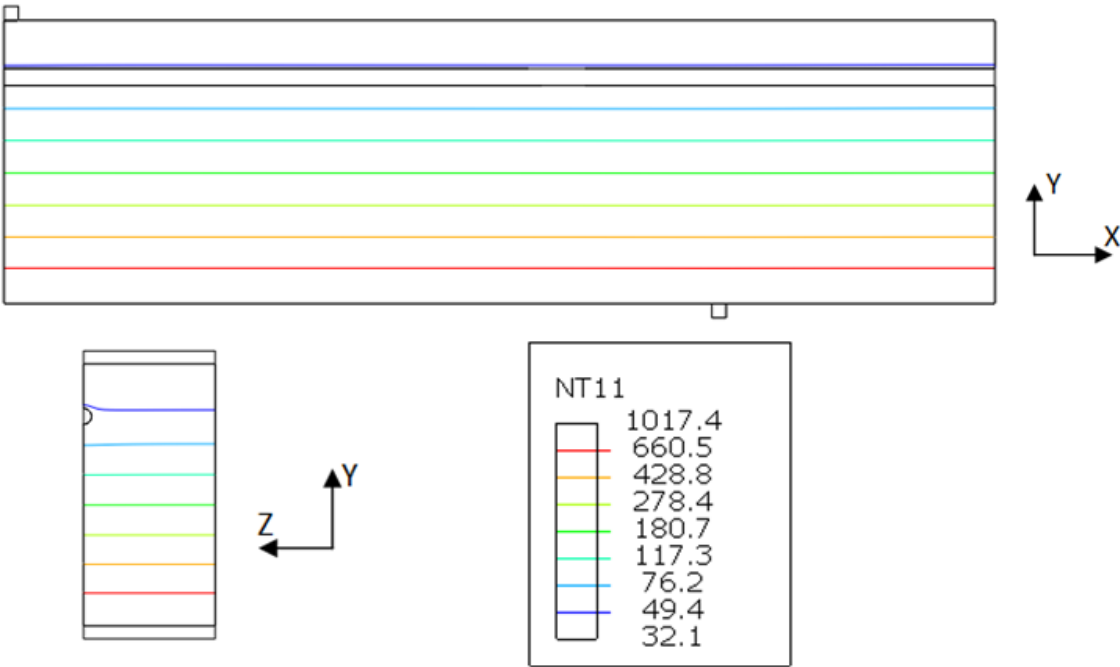


Figure 6.14: Temperature contour plot for 200mm deep slab with 12mm diameter bar at 120 minutes fire duration, values in °C

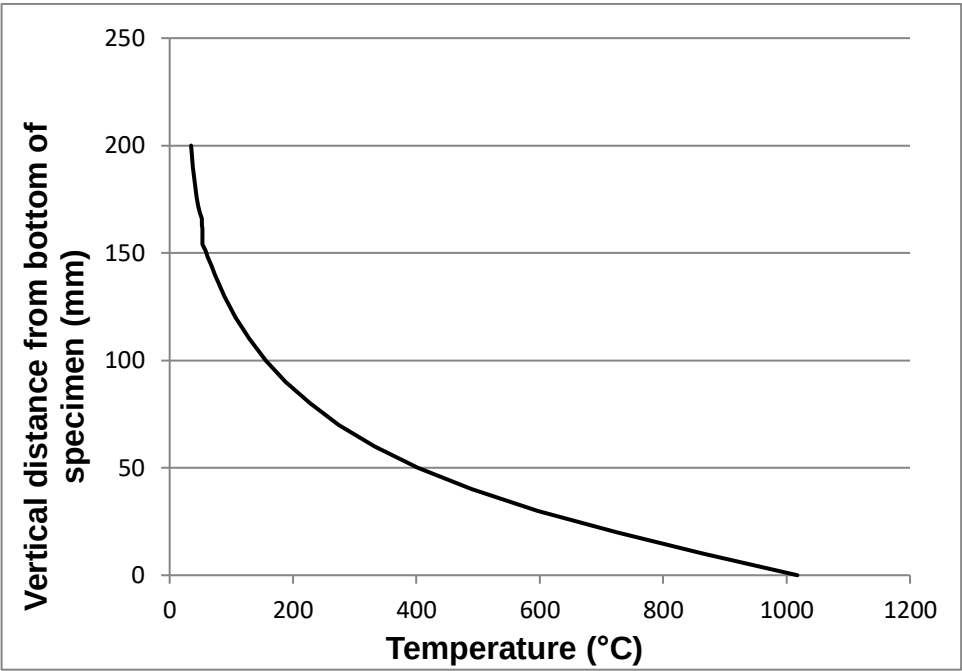


Figure 6.15. Temperature variation with distance from the bottom of the slab for the 200mm deep slab with 12mm diameter bar at 120 minutes fire duration at the location shown in figure 6.11.

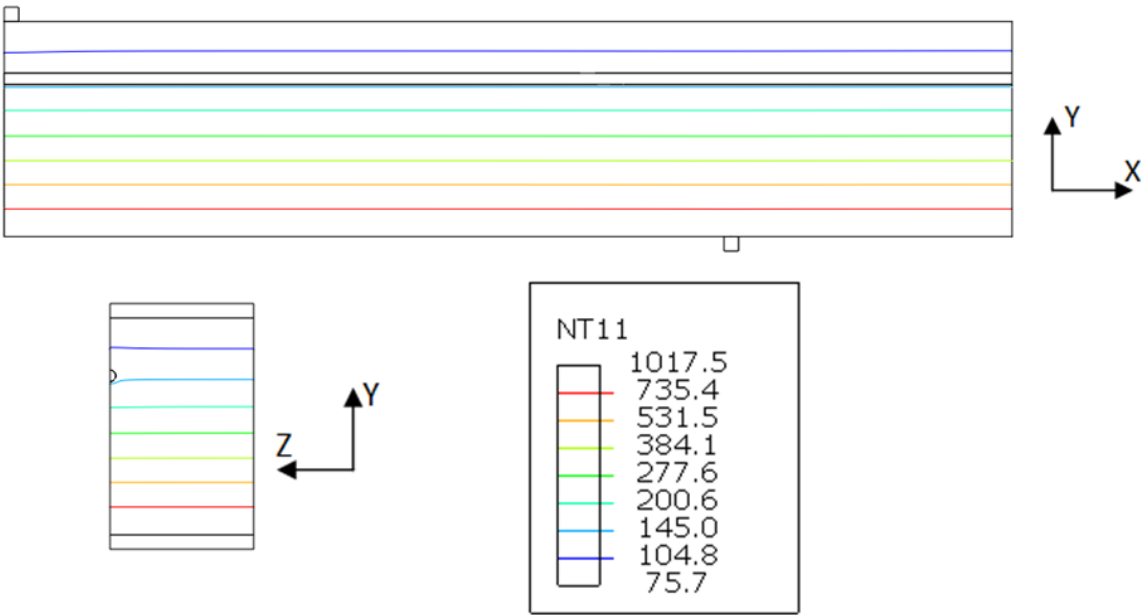


Figure 6.16: Temperature contour plot for 150mm deep slab with 8mm diameter bar at 120 minutes fire duration, values in °C

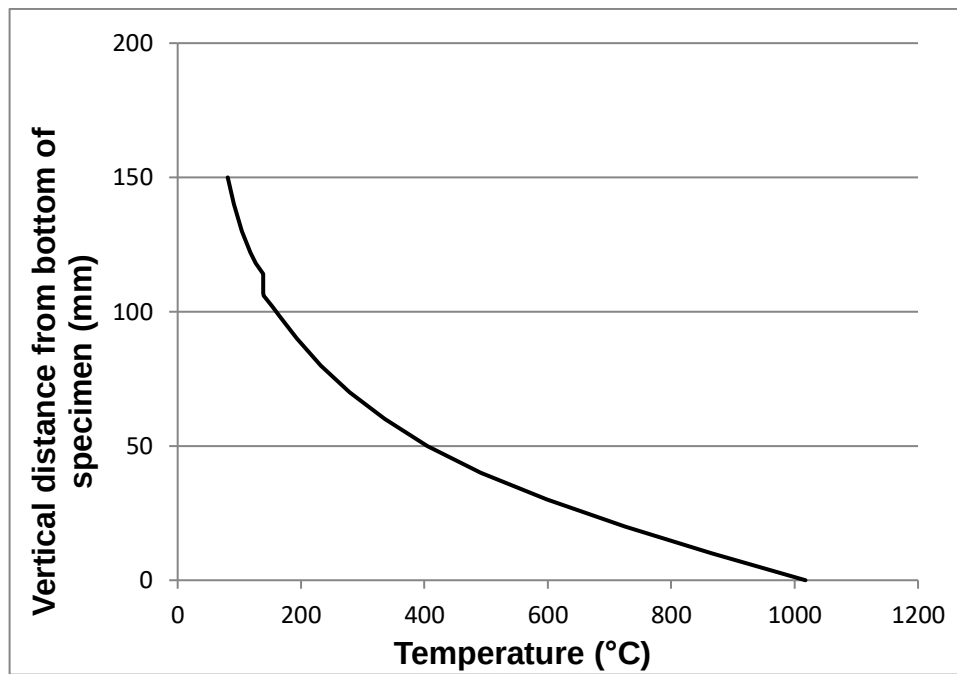


Figure 6.17. Temperature variation with distance from the bottom of the slab for the 150mm deep slab with 8mm diameter bar at 120 minutes fire duration at the location shown in figure 6.11.

The different bar diameters have very little influence on temperature distribution throughout the slab. The concrete temperature in the lower part of the slab is much higher than the upper parts, reaching a maximum of around 1017°C for all slabs considered. The average temperatures of the concrete and steel were significantly higher in the shallower slab than the deeper one, but the concrete temperature in the lower part of the slab was very similar for different slab depths.

At 120 minutes fire duration, the average temperature at the bond interface and throughout the bar was 53°C for the 200mm deep slabs regardless of bar diameter. The equivalent temperature at the bond interface and bar for the 150mm deep slab was 138°C. These bond and steel temperatures are substantially lower than the uniform temperatures of 200°C and 300°C which were used in some of the structural scenarios.

The thermal gradient is included in some simulations to provide a reasonable representation of temperatures in fire scenarios with different durations for the purposes of the comparative study of the reinforcement. There is no expectation or requirement for these temperatures to predict the thermal gradient in any specific real slab with a high level of accuracy. The thermal gradient in the slabs is in reasonable agreement with the values given in Eurocode 4 part 1-2 (BSI, 2005a) as an approximation of temperatures at different depths within a concrete slab with exposure to ISO standard fire. This demonstrates that the thermal gradients are reasonable and sufficient for the purposes of this research.

7 Results, Analysis and Discussion of Numerical Modelling Research

This chapter presents the results, analysis and discussion of the finite element study. The results and analysis relating to specific parameters of interest are separated into different sub-sections of chapter 7. The discussion relating to each of these parameters is also included in sub-sections within chapter 7, following the relevant results and analysis. The only aspect of tensile membrane action development which is being considered here is the behaviour of the reinforcement in the cracked region around the perimeter supports of the slab.

The results are presented graphically in three different forms:

- The first form is a plot of bond slip against distance along the bar, at the point of ultimate strain in the bar. The bond slip was attained from the difference in displacement in the X direction of each node along the surfaces of the bar and concrete. The position on the cross section at which these X direction displacements were measured is shown in figure 7.1. These nodes were initially aligned. The distance is the initial, non-deformed, distance in the X-direction, as indicated by the yellow arrow in figure 7.2. This was measured at the surfaces of the bar and concrete at half-height along the bar.
- The second form is a plot of axial strain against distance along the bar, to either side of the point of ultimate strain along the bar. This was measured through the centre of the bar, as indicated in figure 7.1.
- The third form of plot is of vertical load against vertical deflection. These values are measured at the location at which controlled displacement is

applied to the model, on the upper bearing plate shown in figure 7.2. Therefore, these are not expected to be values of overall load or deflection on a slab, but they do provide a consistent benchmark for comparison between the different scenarios considered

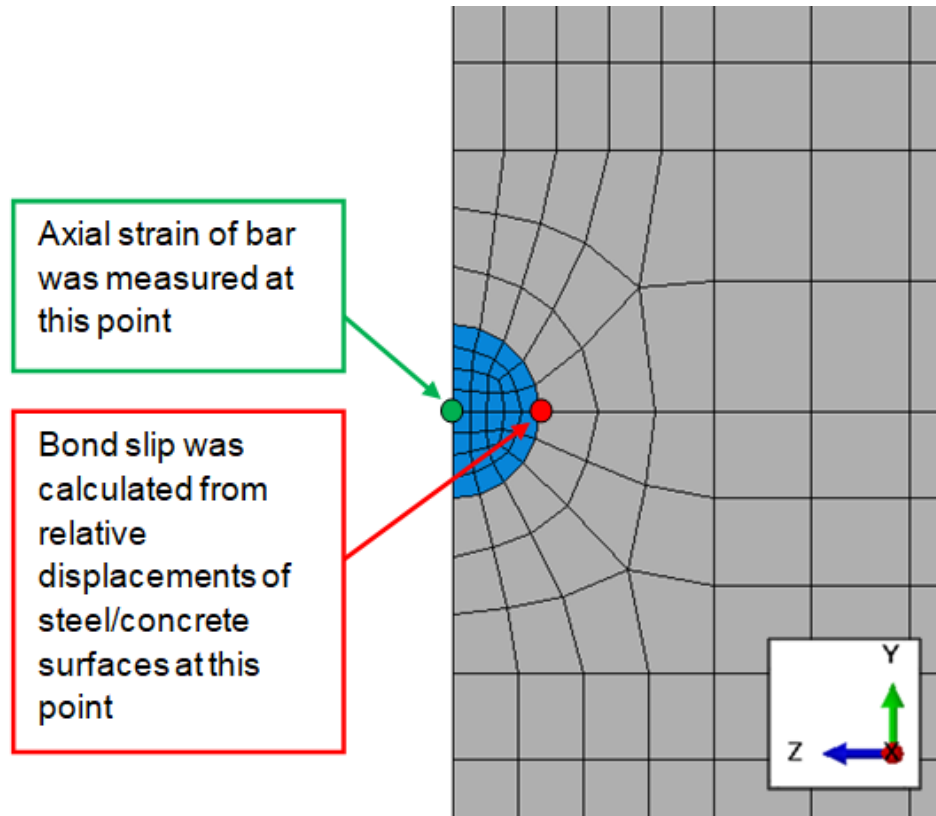


Figure 7.1. Annotated partial view of model showing locations on cross-section at which axial strain and axial displacements were measured.

The first two of these plots are useful because they demonstrate the extent to which the bar has slipped and how localised the strain in the bar has been at the point of rupture. The effect of each parameter on bond slip and strain localisation can be deduced by comparing these outputs between simulations with different parameters. Greater bond slip is clearly beneficial in allowing reinforcement to span a crack without rupture as it reduces the localisation of strain.

The load-deflection plots are useful primarily because they demonstrate how much vertical deflection the specimen was able to withstand. If a specimen has a greater ability to withstand vertical deflection before reinforcement rupturing, the slab which this specimen represents will be more capable of facilitating the rotation of the slab which happens near the boundaries as tensile membrane action develops.

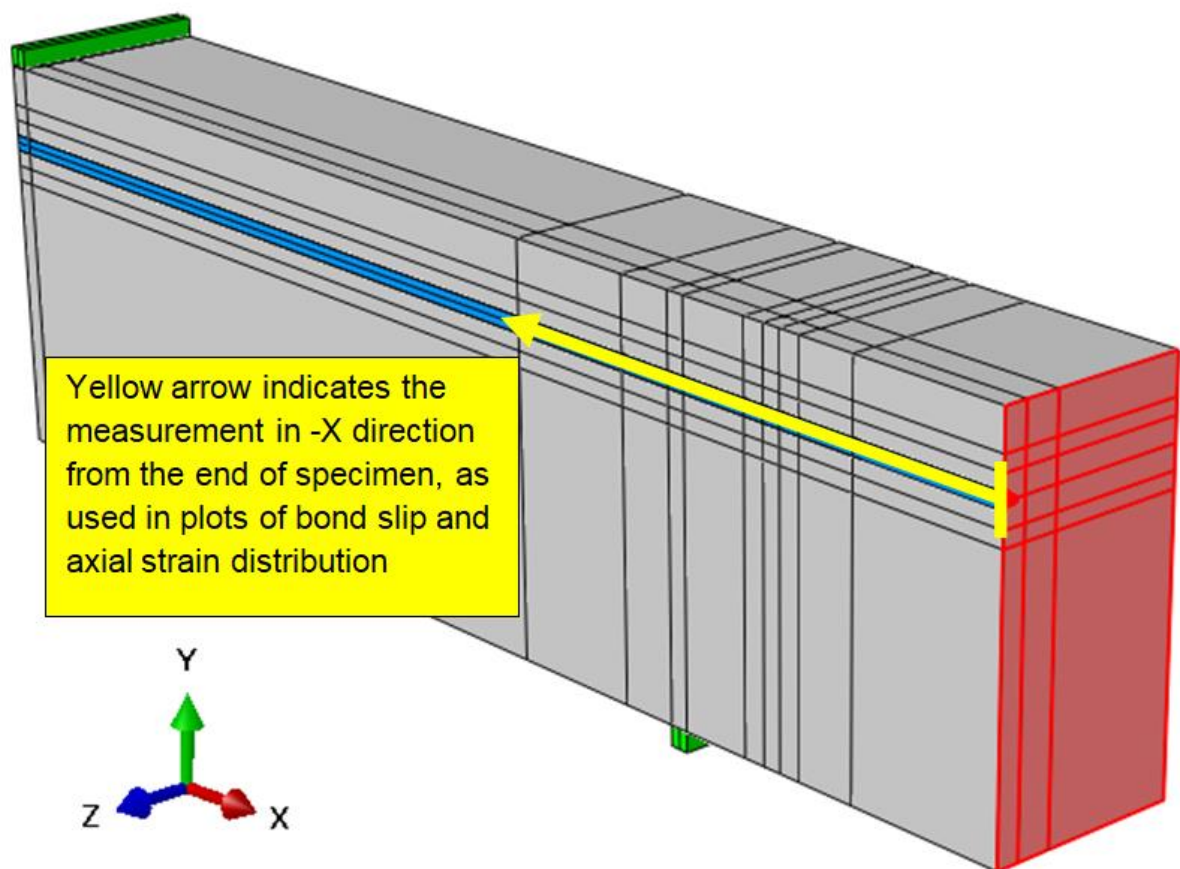


Figure 7.2. View of model showing the face (highlighted red) from which distances were measured in the axial strain and bond slip plots in chapter 7.

7.1 Explanation of the overall results

The intention of this section is to explain some of the overall behaviours which are presented in the results. The results of the 8mm diameter deformed bar at ambient temperature have been presented in this section. These results will be compared with the results of other simulations in further sections, and these comparisons are more meaningful in terms of the research objectives. This single set of results has been used in this section as an example to explain the significant features which are common features throughout most of the results.

Figure 7.3 shows a view of the model at the point of rupture in the bar, with a contour plot of axial strain in the global X direction. Mesh is omitted for clarity of the bar rupture position. This figure has been included here to demonstrate the overall response of the modelled specimen, which was similar for all simulations. The crack development is represented by large and highly concentrated strain due to localised tensile damage which progressed downward from the top surface during the simulation, and is clearly visible in figure 7.3. The location of the crack was not predefined, but it did develop in this location in all simulations except for one, which is presented in section 7.9. The location of the rupture is visible in figure 7.3, and is located within the region of the simulated crack.

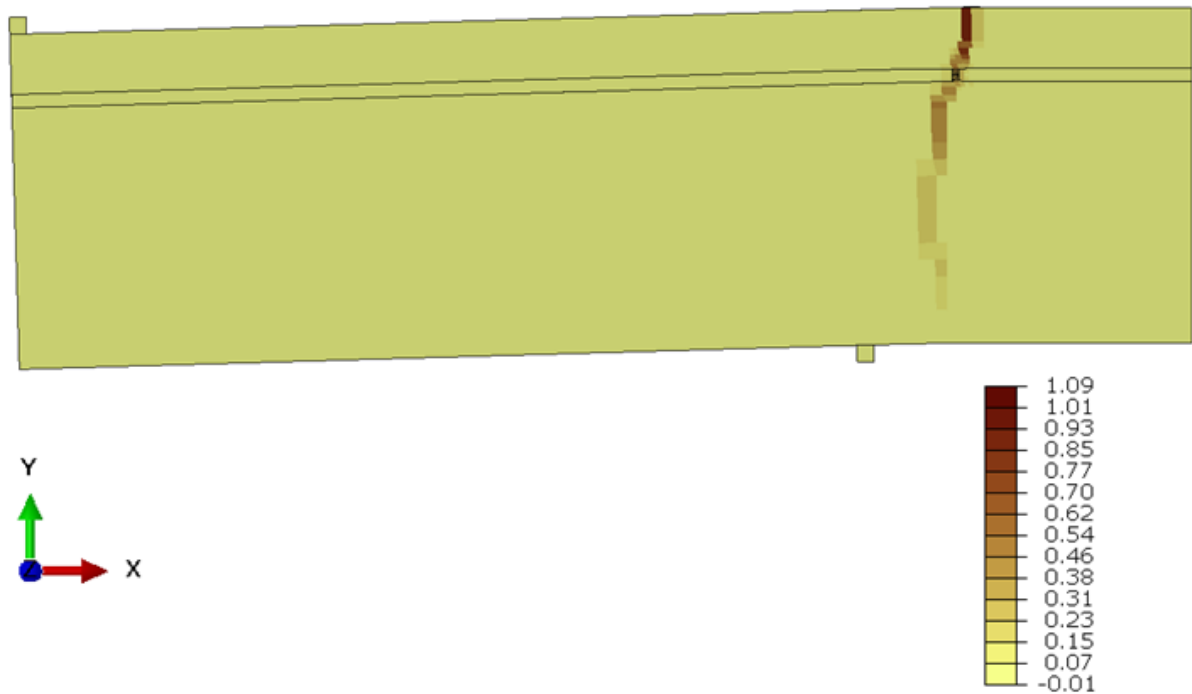


Figure 7.3. View of simulation output for 8mm diameter deformed bar at ambient temperature, showing contours of axial (X direction) strain at the stage of reinforcement rupture.

Figure 7.4 shows a plot of bond slip with respect to distance, where distance is the initial distance, measured in the -X direction in the axis convention that has been used consistently throughout chapters 6 and 7. This distance was explained at the start of chapter 7, and is the distance from the surface highlighted red in figure 7.2. This distance is intended to represent the distance along the reinforcing bar to the centre-line of a downstand beam which supports the slab, as shown in figure 6.1. Figure 7.4 shows the distribution of slip at the point of ultimate strain in the bar.

As evident from figure 7.4, the slip increases sharply in the region in which the crack developed. The slip reverses direction at the crack because the bar slips in opposite directions on either side of the crack. The slip also reduces further away from the crack. The slip was limited enough in all simulations considered that the bond did not

reach peak bond stress before rupture of the bar. Bond slip is beneficial for reinforcement spanning a crack as it will allow the strain to be less concentrated.

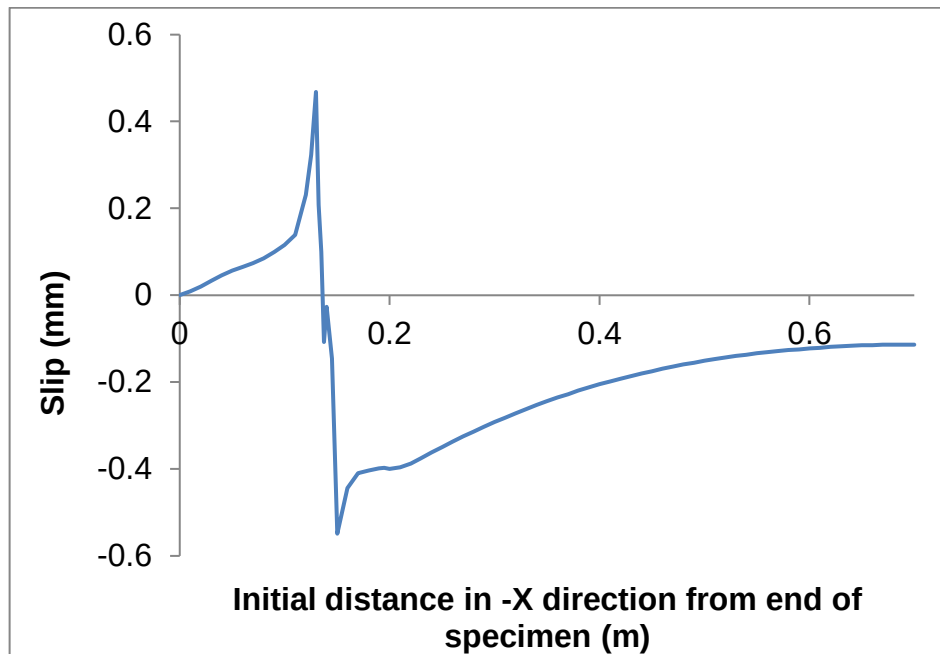


Figure 7.4. Plot of bond slip with respect to distance measured along the reinforcing bar for 8mm diameter deformed bar

Figure 7.5 shows axial strain relative to distance, where distance is measured the same way as in figure 7.4. This is the axial strain distribution at the point of ultimate strain in the bar. The yield strain, ε_{ay} , the limiting strain for yield strength, ε_{au} , and the ultimate strain, $\varepsilon_{ae,\theta}$, are shown on figure 7.5. The strain, ε_{au} , is the strain at which point steel stiffness damage is onset in the model and the strength of the steel reduces linearly until rupture at $\varepsilon_{ay,\theta}$, as described in section 5.3.

As the axial strain distributions for each simulation are shown in this type of figure at the point where the ultimate strain reaches $\varepsilon_{ae,\theta}$, they will all peak at 0.2. The axial strain peaks where the crack formed. This demonstrates that strain is concentrated in the region of the crack. The important feature of these strain plots is the width of the spike as this is a measure of how concentrated the axial strain was in the bar at

the point of rupture. In scenarios where the peak is wider, the strain is less concentrated and reinforcement will be more capable of spanning a crack without rupturing.

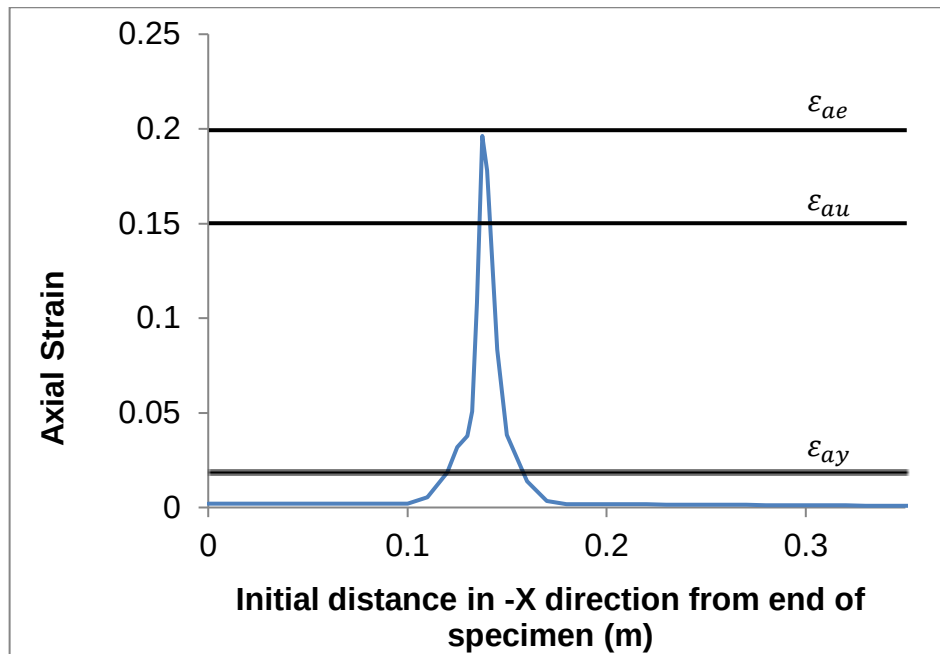


Figure 7.5. Plot of axial strain with respect to distance measured along the reinforcing bar for 8mm diameter deformed bar

Figure 7.6 is a load-deflection plot, where the load is the vertical load due to the controlled displacement of the upper bearing plate in the model. The deflection is measured from the same point. Some important features of this curve have been described on this curve in figure 7.6. The shape of the initial peak in vertical load is due to the tensile strength and stiffness of the concrete as tensile damage begins to develop within the model. The peak is very pronounced due to the very low reinforcement ratio of the cross section.

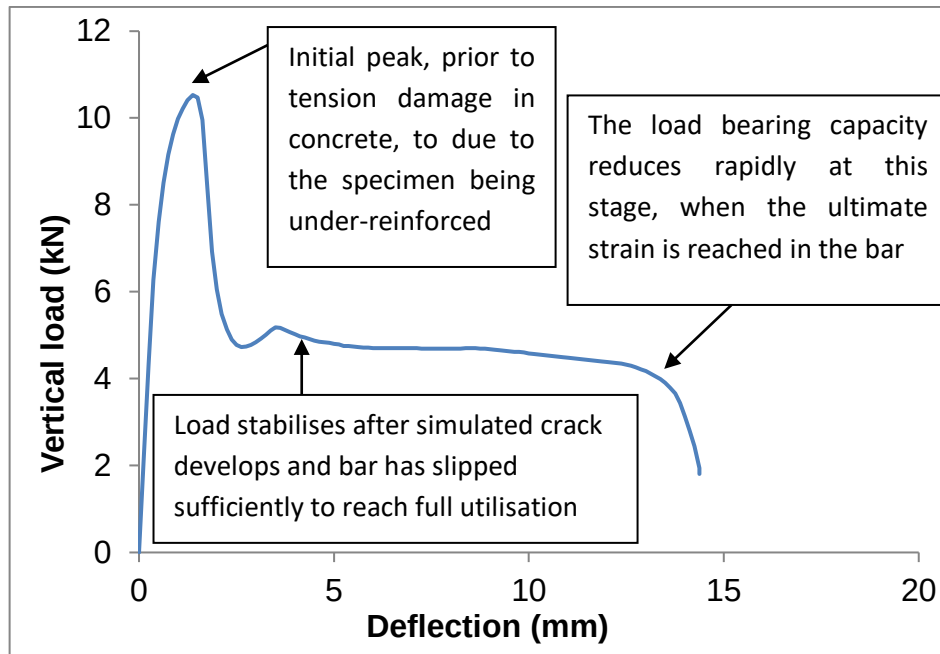


Figure 7.6. Load-deflection plot for 8mm diameter deformed bars at 20°C uniform temperature

The slip, strain and load-deflection curves are not very meaningful in isolation, but the comparisons between these curves for different simulations demonstrate the relative performance of crack-spanning reinforcement in different scenarios. Greater slip in the bond slip plots, a wider distribution of strain in the strain plots and greater ability to withstand vertical deflection in the load-deflection plots are the features which demonstrate that the conditions of one simulation are more favourable than the conditions of another simulation. These are the key features to observe in the plots in the following sections.

7.2 Mesh size sensitivity study results and analysis

A mesh size sensitivity study was conducted to confirm the suitability of the mesh size adopted. It was necessary to present this mesh sensitivity study at this stage of the thesis because section 7.1 provides context to the level of mesh size sensitivity and its significance to the research outcomes. Figures 7.8, 7.9 and 7.10 below are the equivalent plots to those shown for the other simulations throughout chapter 7 but showing the output with different mesh sizes. The sensitivity study was conducted on a model featuring an 8mm diameter deformed reinforcing bar and a uniform temperature of 200°C. The mesh size was changed only in the regions of the finer mesh, where the local behaviour was being studied. The surfaces where the mesh size varied in the sensitivity study are highlighted red in figure 7.7. These variations in mesh size were projected internally through the full depth of the model.

The areas in which the mesh size was varied included the regions which were of interest to the study, including the area around the bond and where the crack developed, with the exception of the one simulation where the crack moved. The results of that one simulation should be considered in context of the crack having developed in a region which was of coarser mesh, although the bonded region is of more significance.

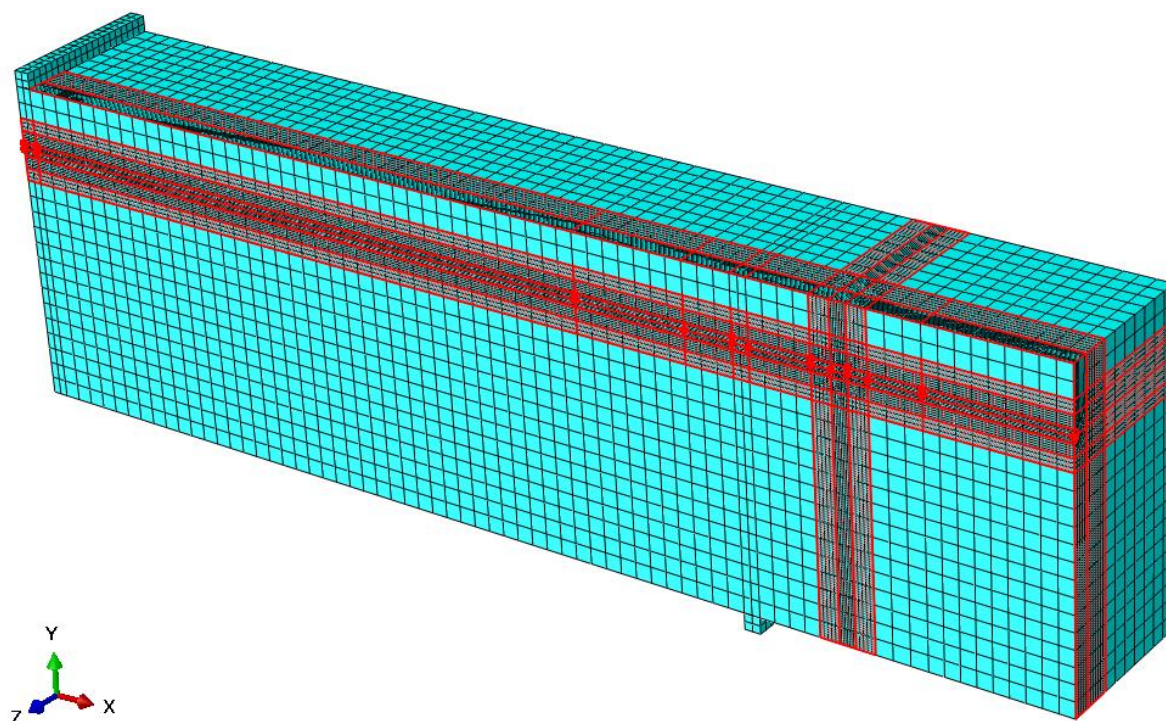


Figure 7.7. Surface regions in which mesh size was varied for sensitivity study
(highlighted red)

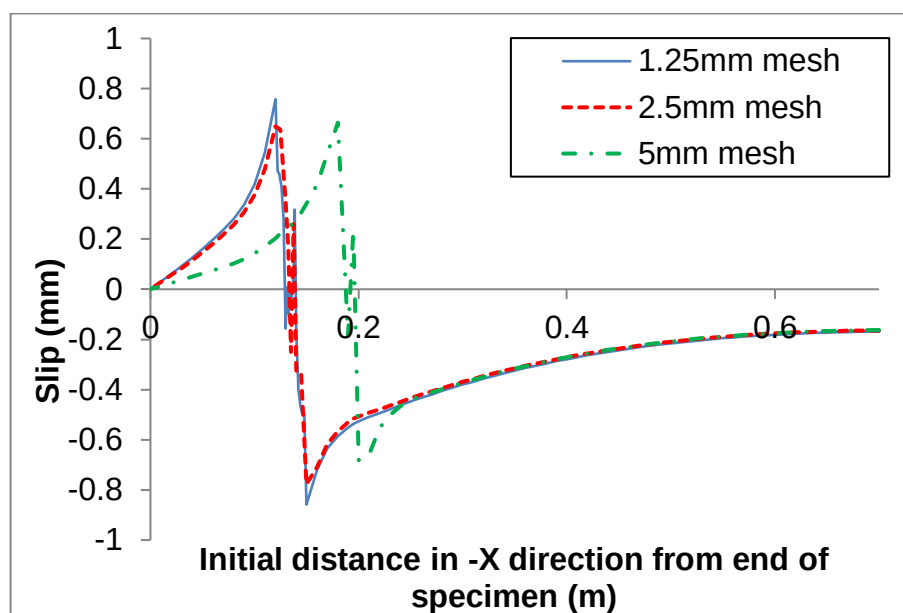


Figure 7.8. Plot of bond slip with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bars at 200°C uniform temperature, with different minimum mesh sizes

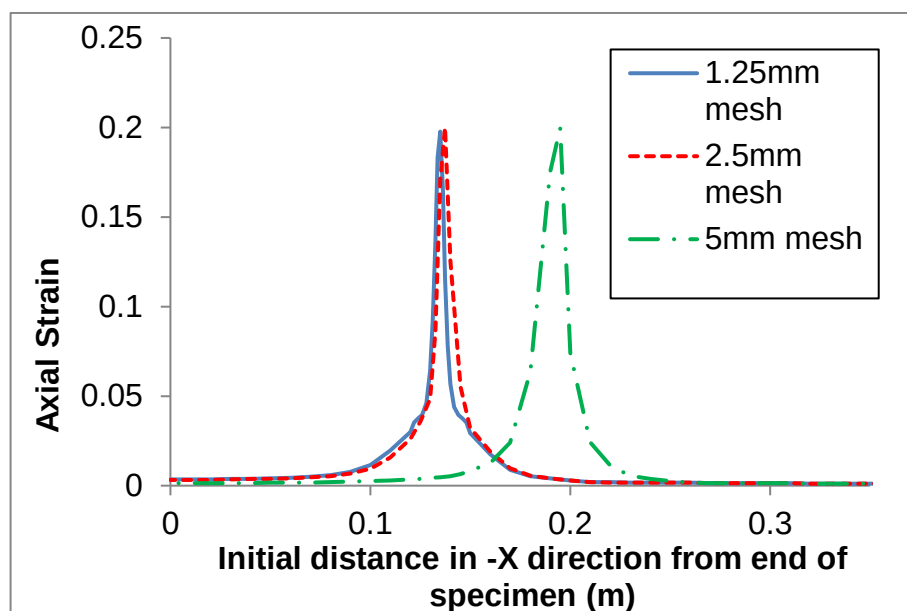


Figure 7.9. Plot of axial strain with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bars at 200°C uniform temperature, with different minimum mesh sizes

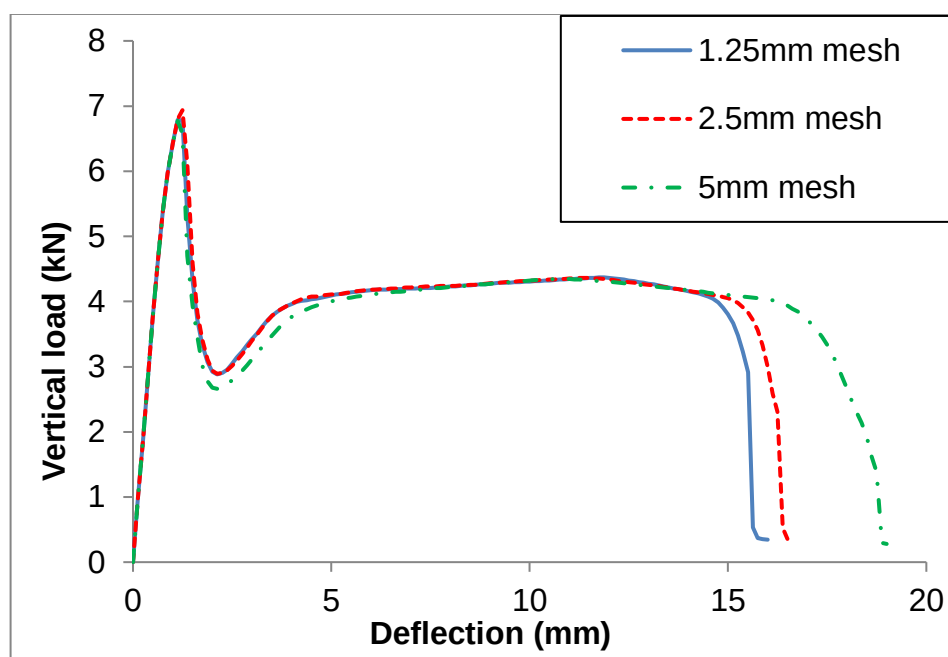


Figure 7.10. Load-deflection plot for 8mm diameter deformed bars at 200°C uniform temperature, with different minimum mesh sizes

From figures 7.8 and 7.9, it is evident that the cracking and rupture behaviour of the slab has been altered by increasing mesh size from 2.5mm to 5mm. The crack location moved approximately 70mm further away from the centreline of the support. As shown in figure 7.10, the vertical deflection at failure also increased by approximately 2mm with increased mesh size. With 5mm mesh relative to 2.5mm mesh, the difference in vertical deflection at the critical point at which damage onset in the bar was over 10%.

The slightly increased peak slip values in figure 7.8 for the 1.25mm mesh, relative to the 2.5mm mesh can be explained by the increased spatial resolution of outputs due to the greater concentration of nodes in this region and is not significant in terms of the research outcomes. With 2.5mm mesh relative to 1.25mm mesh, the difference in vertical deflection at the critical point at which damage onset in the bar was only around 3%, as shown in figure 7.10. This difference is small in the context of the level of accuracy which could be expected from the model, given the level of uncertainty in the inputs. Furthermore, this difference is not large enough to meaningfully change any of the research outcomes in the following sections. In summary, the use of 2.5mm mesh is justified as it reduces computational time and file size without meaningfully changing the research outcomes. Therefore, 2.5mm is used as a typical size for finer meshed regions for the remainder of this study.

7.3 Results and analysis - fire durations

This section presents the results of three simulations which were performed using 8mm diameter deformed bar at 20°C and after fire durations of 60 minutes and 120 minutes. In these, and all other simulations with different fire durations, the displacement was increased at the end of the fire duration, and not throughout the duration. These simulations were numbered 1,2 and 3 in table 6.2. The results of these simulations have been compared in order to investigate the effect of increased fire duration on the behaviour of crack-spanning reinforcement around the boundaries of a slab as tensile membrane action develops. These results are discussed in section 7.5.

At the point of rupture, the distribution of bond slip in the slab is similar after the different fire durations, as evident from figure 7.11. The extent to which strain in the bar is localised and concentrated in the cracking region is similar for the different fire durations, as evident from figure 7.12. The reason for the similar bond slip and strain localisation after different fire durations is explained in section 7.5. With increased fire duration, the slab is able to undergo greater vertical deflection, and therefore rotation, before damage onsets in the bar and the vertical load carrying capacity rapidly reduces, as shown in figure 7.13. The load carrying capacity of the slab reduced with increasing fire duration.

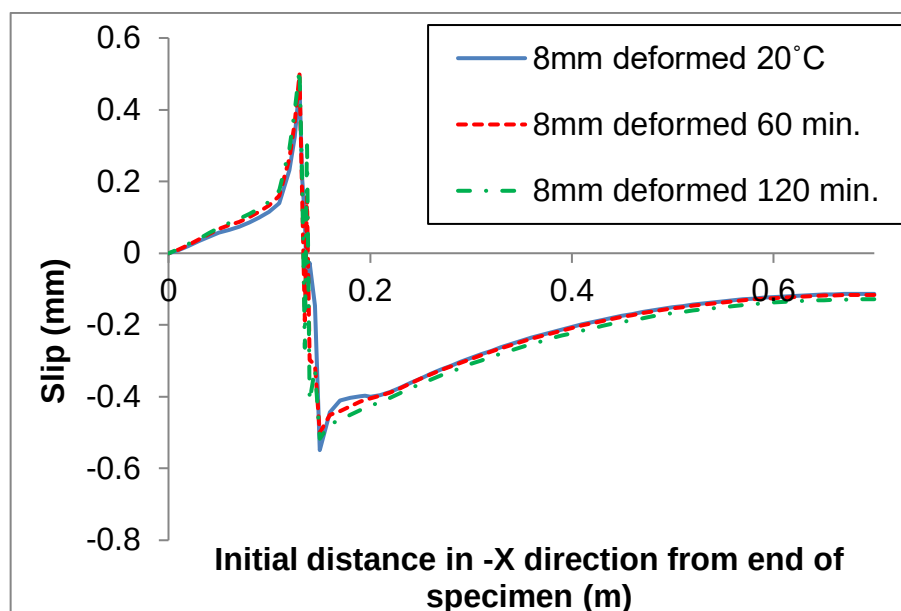


Figure 7.11. Plot of bond slip with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bar with different fire durations

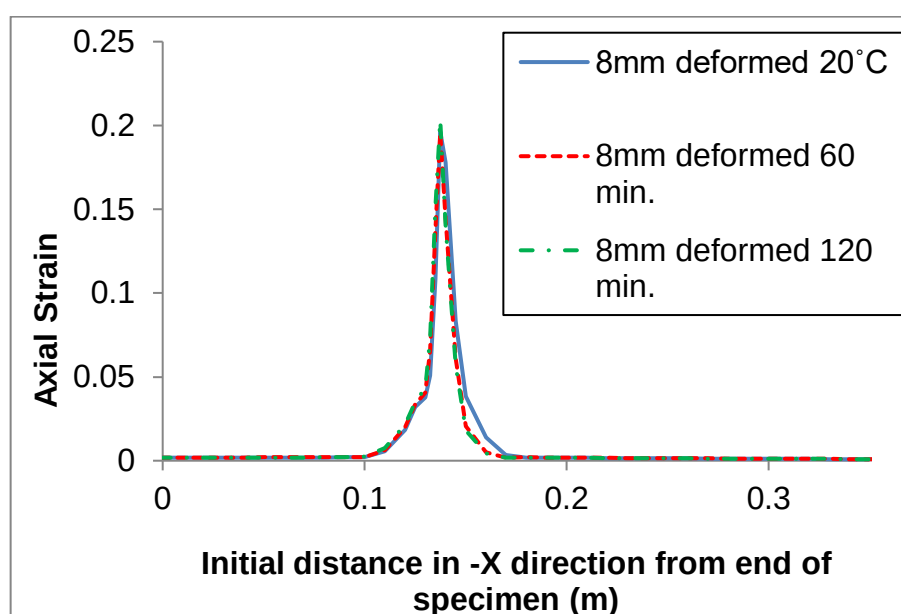


Figure 7.12. Plot of axial strain with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bar with different fire durations

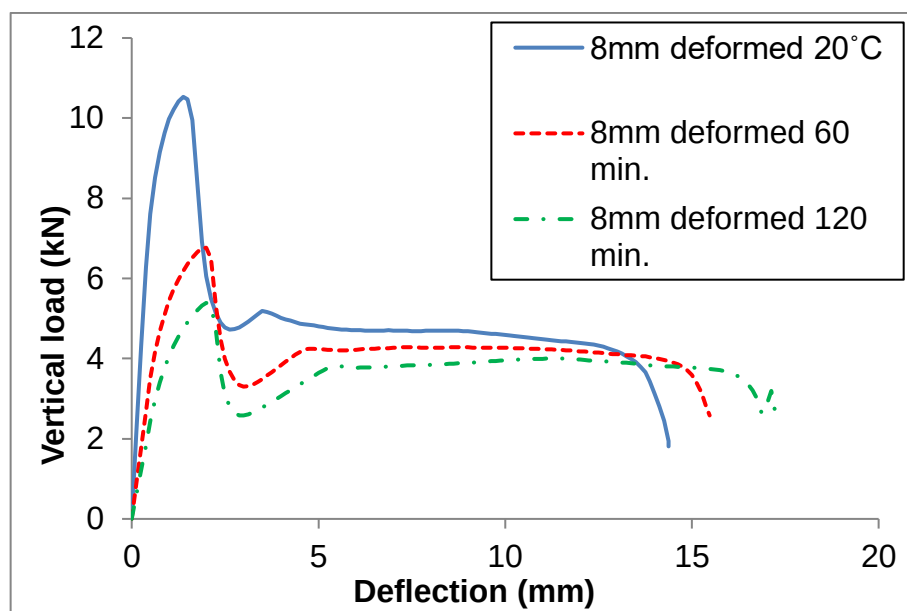


Figure 7.13. Load-deflection plot for 8mm diameter deformed bar with different fire durations

7.4 Results and analysis - uniform temperatures

This section presents the results of three simulations which were performed using 8mm diameter deformed bar with 20°C, 200°C and 300°C uniform temperatures throughout the specimen. These simulations were numbered 1,4 and 5 in table 6.2. Comparing the results of these simulations is useful as it demonstrates the effect of temperature on the behaviour of reinforcement at perimeter crack locations as tensile membrane action develops in a fire scenario. These results are discussed in section 7.5.

At the point of bar rupture, there was increased slip in the cracked region with increased uniform temperature, as is shown in figure 7.14. At the same point, the axial strain in the bar is less localised and concentrated in the crack region with increased uniform temperature, as evident from figure 7.15. As with increased fire duration, with increased uniform temperature the slab withstands greater vertical displacement within the temperature range considered, as shown in figure 7.16. As expected, the strength of the slab reduces with increasing uniform temperature.

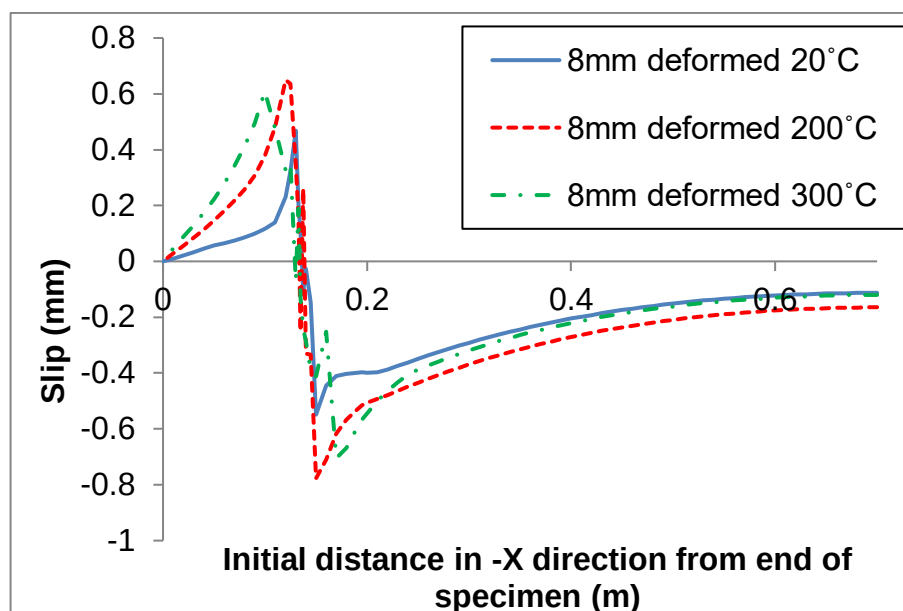


Figure 7.14. Plot of bond slip with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bar with different uniform temperatures

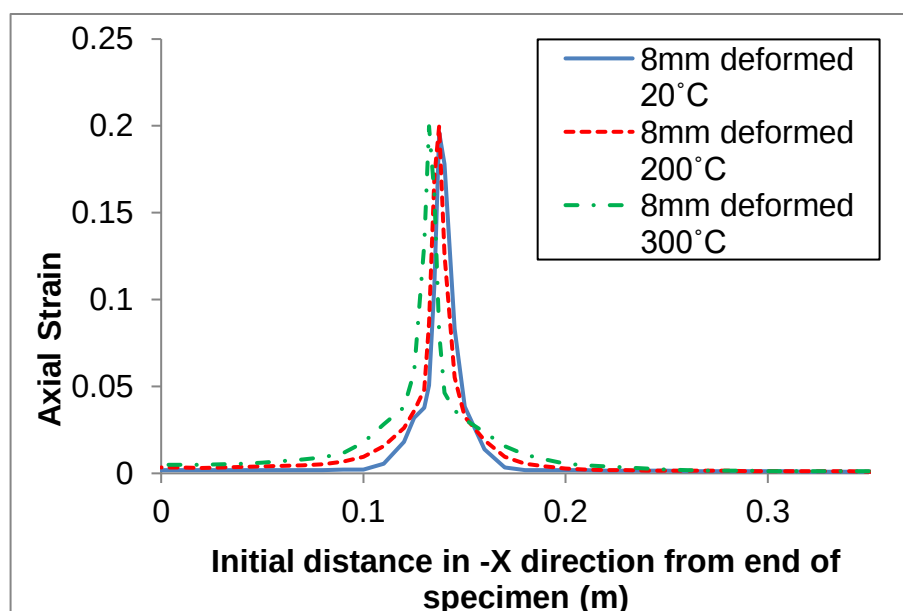


Figure 7.15. Plot of axial strain with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bar with different uniform temperatures

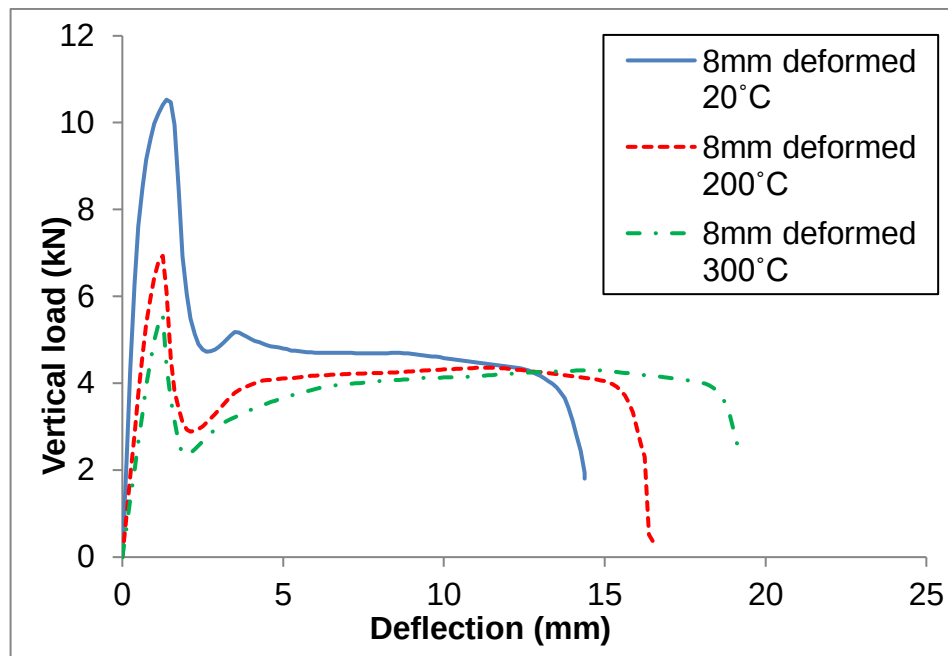


Figure 7.16. Load-deflection plot for 8mm diameter deformed bar with different uniform temperatures

7.5 Discussion on the effect of elevated temperature

The purpose of this research is to investigate the behaviour of the crack-spanning reinforcement at elevated temperature in the situation of tensile membrane development. As the crack opens, there will be a concentration of strain in the reinforcement where it spans the crack. The concentration of strain is reduced by bond slip as this increases the length over which the bar must extend to the opening of the crack. It is important to note that figures 7.11, 7.12, 7.14 and 7.15 are all at the point when ultimate strain is reached in the bar in each simulation and this occurred at different stages in each simulation.

There was very little difference between the amount of bond slip after the different fire durations considered, as shown in figure 7.11. This is because the temperature at the bond interface was only 53°C after 120 minutes fire duration in these

specimens and the bond stiffness will be very similar to its ambient temperature state. As neither the bond or the steel material properties are significantly influenced by this temperature difference, the extent of strain localisation is very similar for the simulations after different fire durations, as shown in figure 7.12.

The load-deflection response was affected by these fire durations, as shown in figure 7.13. The difference is due to the high temperatures in lower regions of the slab reducing the concrete strength and stiffness. This weakens the overall flexural response and the flexural neutral axis is moved higher in the specimen, which moves the point of rotation at the crack higher. This means that there is more rotation in the slab before the crack width reaches the width that causes rupture of the bar. As the specimen is weaker in flexure but tolerates more rotation, the load carrying capacity is reduced but the ability of the slab to withstand vertical displacement is increased, as shown in figure 7.13.

The higher temperatures in the lower regions of a slab due to increased fire duration will increase the ability of a slab to facilitate the rotation which occurs around the perimeter of the bay during the formation of tensile membrane action. This happens due to changes in concrete and the overall flexural response, reducing the crack width, rather than changes to the stiffness of the bond.

With higher uniform temperature in the specimen, the bond slip at the point of rupture did increase moderately, as shown in figure 7.14. The strain also became less concentrated at uniform elevated temperatures than at ambient temperature, as shown in figure 7.15. This is due to the higher temperature at the bond reducing its stiffness, but only modestly. Figure 7.17 is a plot of the 8mm deformed bar stress-

slip curves from the experiments described in chapter 3, annotated to demonstrate the relevant aspect of elevated temperature bond behaviour.

The initial peak in load carrying capacity is not of particular interest to this research, but it is reduced due to the reduction in concrete tensile strength at these temperatures. As evident from figure 7.16, the load carrying capacity of the specimens is quite similar at different uniform temperatures after the initial peak at around 5 to 10mm deflection, where the bar has slipped sufficiently to be fully utilised. This is due to the mechanical properties of the concrete in compression and the steel being relatively unaffected by these temperatures. The difference in ability to withstand vertical deflection between specimens is primarily due to the effect of temperature on the bond response in these simulations as temperatures are not high enough to substantially change the compressive properties of concrete.

The most important point of this discussion is that these fire durations do not significantly change the stiffness of the bond response, but the higher temperatures in the compressive region of concrete change the flexural response such that the slab can withstand greater rotation at the perimeter of the bay with the same crack width at the position of the bar. Therefore, increased fire duration, at least up to 120 minutes, is beneficial in allowing a slab, of this depth, to develop tensile membrane action without rupture of the crack-spanning reinforcement near the perimeter of the bay.

As previously explained, the model is of a slab with flat profile, rather than a composite slab with a profiled steel deck. Although the steel deck of a composite slab will weaken in the fire scenario to the extent that it can be considered to have negligible strength or stiffness (Bailey et al., 2000; Jiang et al., 2022), composite

slabs do often have a trapezoidal cross sectional profile. The different shape of a composite slab will result in a different temperature distribution in the slab, and there may be less cover to some of the hogging reinforcement bars. It is logical to expect that a slab with less concrete cover from the exposed lower surface to the hogging reinforcement would have higher temperatures in the reinforcement. In such conditions it would be reasonable to expect that bond of the bar would become less stiff, reducing the concentration and localisation of strain in the reinforcement bar, as was observed with the 300°C uniform temperature simulation. These considerations are similar to those of reduced slab depths, which are considered further in sections 7.9 and 7.10.

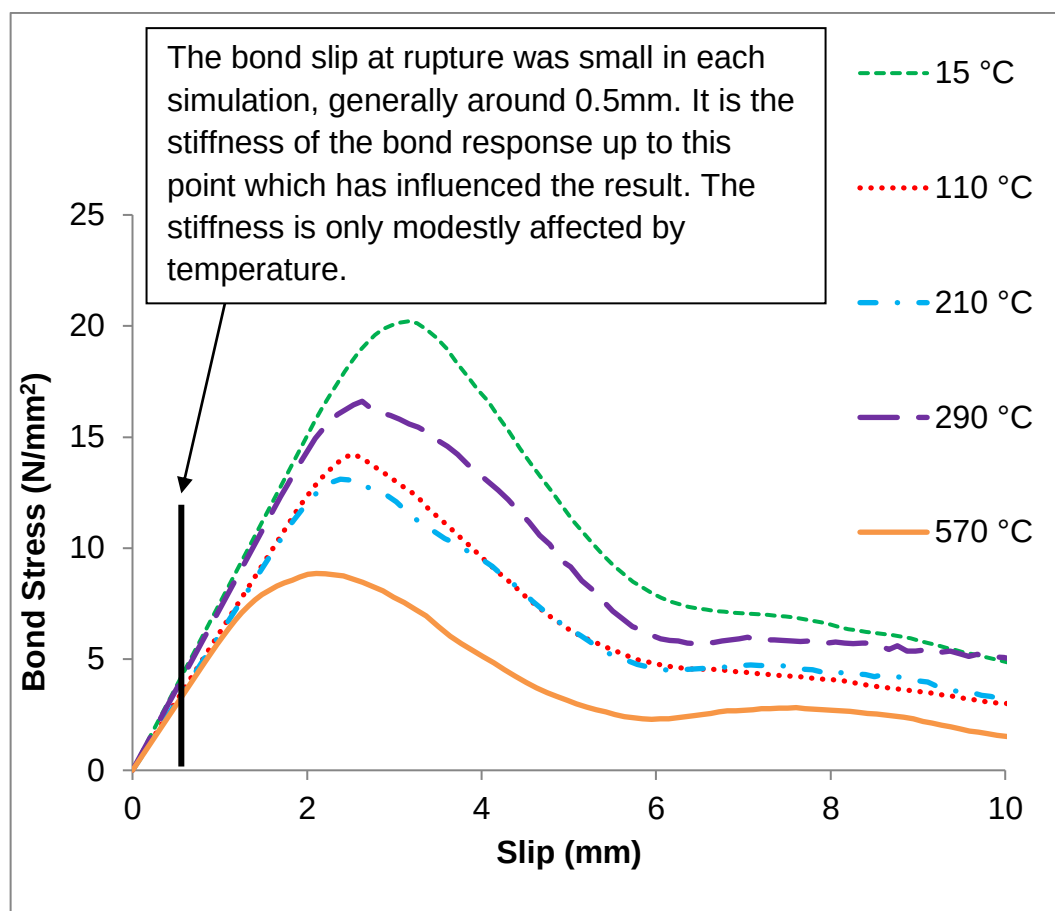


Figure 7.17. Annotated plot of 8mm deformed bar bond stress-slip response at different temperatures from the results of the experiments presented chapter 3

7.6 Results and analysis - bar diameters at ambient temperature

The results presented in this section are for deformed bars of different bar diameter when tested at ambient temperature. These simulations were numbered 1,17 and 19 in table 6.2. The results of these simulations are useful when compared alongside the results in section 7.7 as they demonstrate the effect of bar diameter on reinforcement behaviour at crack locations during the development of tensile membrane action. The comparison of these simulations with those in 7.7 establishes the relevance of the discussion in 7.5 on the effect of elevated temperature to the 10 and 12mm diameter bars. This is discussed in section 7.8.

At ambient temperature, the magnitude of bond slip was similar for the three bar diameters considered, as shown in figure 7.18. The localisation and concentration of strain in the bars reduced, to a small extent, with increasing bar diameter, as evident from figure 7.19.

As evident from figure 7.20, there was little difference in the vertical load bearing capacity between slabs with the different bar diameters at the initial peak because the bars had not yet slipped sufficiently to utilise their full strength. After slip had increased and the bars were fully utilised in terms of strength, the slabs with larger diameter bars had greater capacity, as expected. The vertical deflection at the point of failure was greater with larger bar diameters.

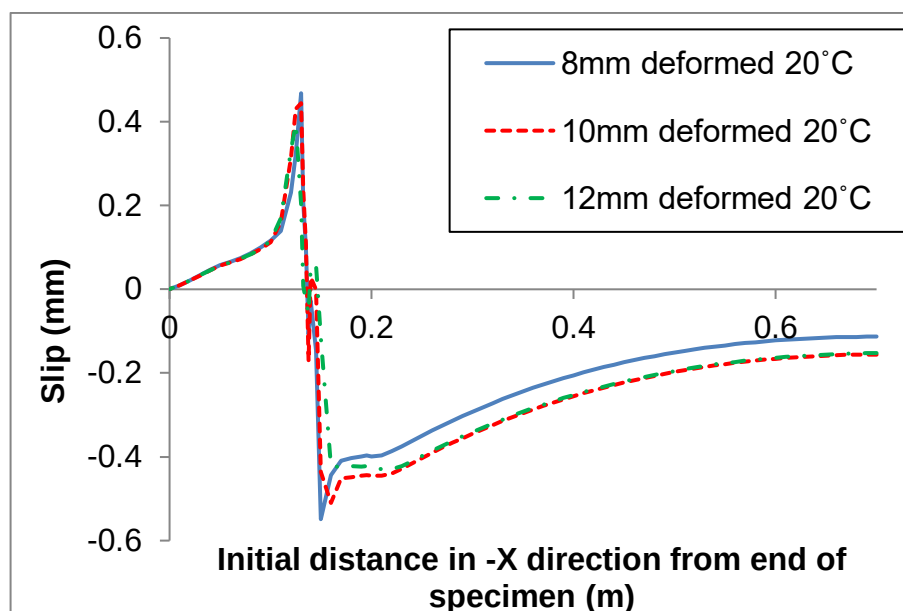


Figure 7.18. Plot of bond slip with respect to distance, measured along the reinforcing bar for deformed bars with different diameters at 20°C

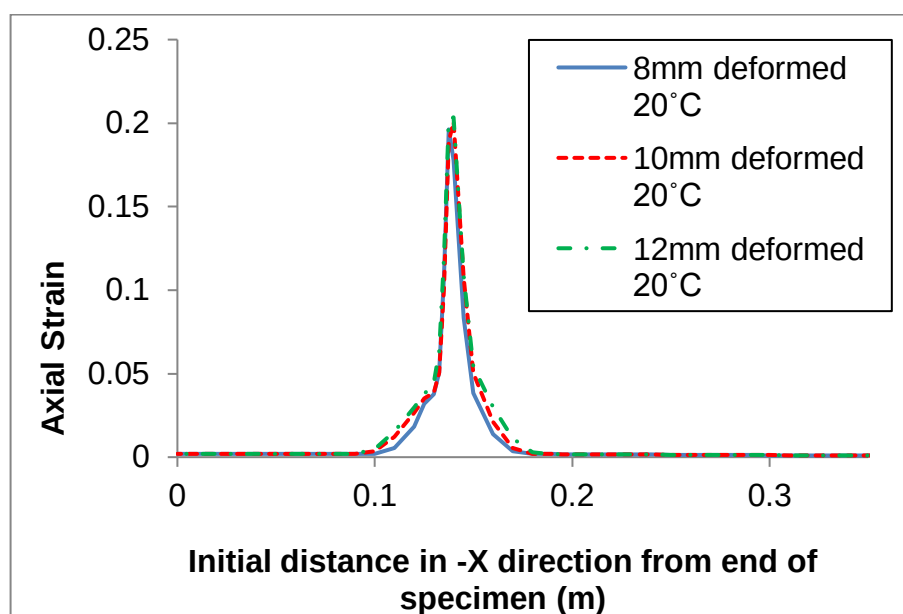


Figure 7.19. Plot of axial strain with respect to distance, measured along the reinforcing bar for deformed bars with different diameters at 20°C

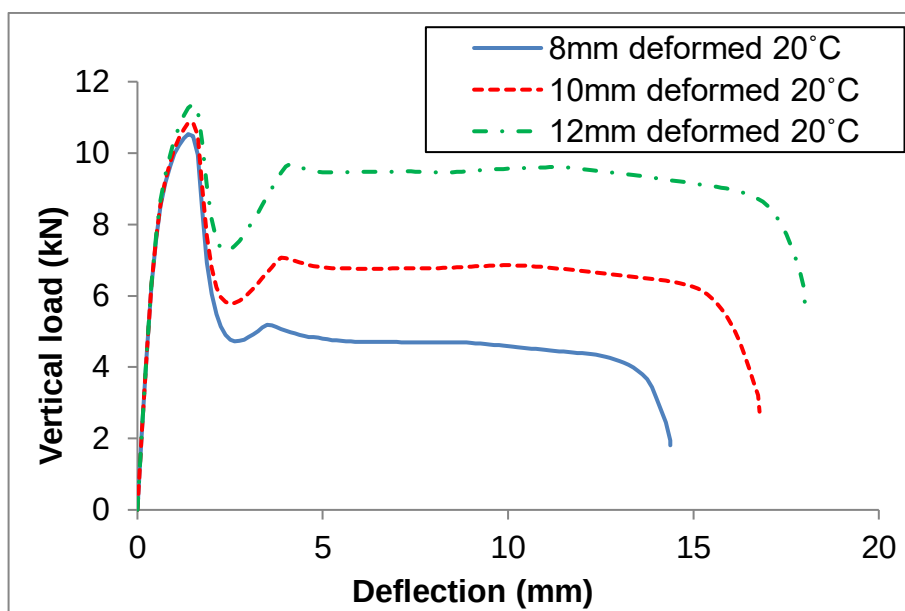


Figure 7.20. Load-deflection plot for deformed bars with different diameters at 20°C.

7.7 Results and analysis - bar diameters after 120 minutes fire duration

The results presented in this section are for deformed bars of different bar diameter when tested after a fire duration of 120 minutes. These simulations were numbered 1,18 and 20 in table 6.2. As with the results at 7.6, the results presented here allow for the effect of bar diameter on reinforcement behaviour during the development of tensile membrane action to be investigated. The results are discussed in section 7.8.

At 120 minutes fire duration, the slip of each bar diameter was similar, but did increase to a small extent with increased bar diameter, as shown in figure 7.21. As at ambient temperature, the localisation and concentration of strain in the bar reduced, to a small extent, with increasing bar diameter, as is evident from figure 7.22.

The slabs with larger bar diameters had greater load carrying capacity after 120 minutes of fire than those with smaller diameter bars, as evident from figure 7.23. After 120 minutes fire duration, the slab with the larger diameter bar withstood greater vertical deflection than the slab with the smaller diameter bar, by a similar margin as at ambient temperature.

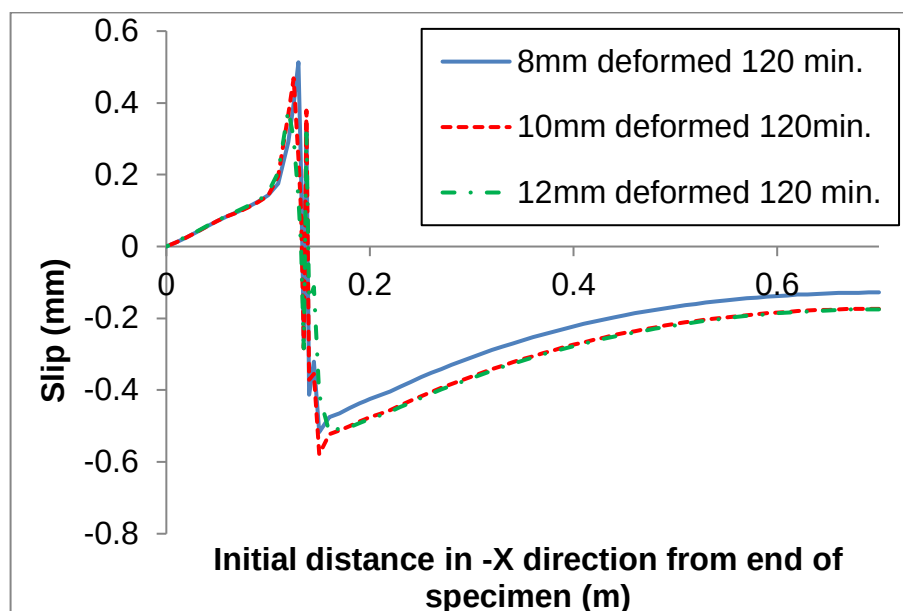


Figure 7.21. Plot of bond slip with respect to distance, measured along the reinforcing bar for deformed bars with different diameters after 120 minutes of fire duration

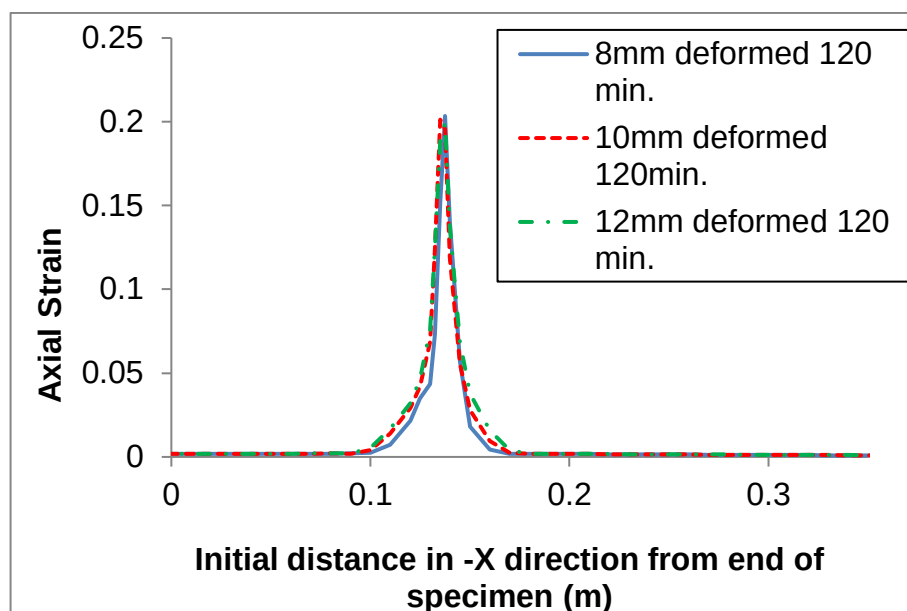


Figure 7.22. Plot of axial strain with respect to distance, measured along the reinforcing bar for deformed bars with different diameters after 120 minutes of fire duration

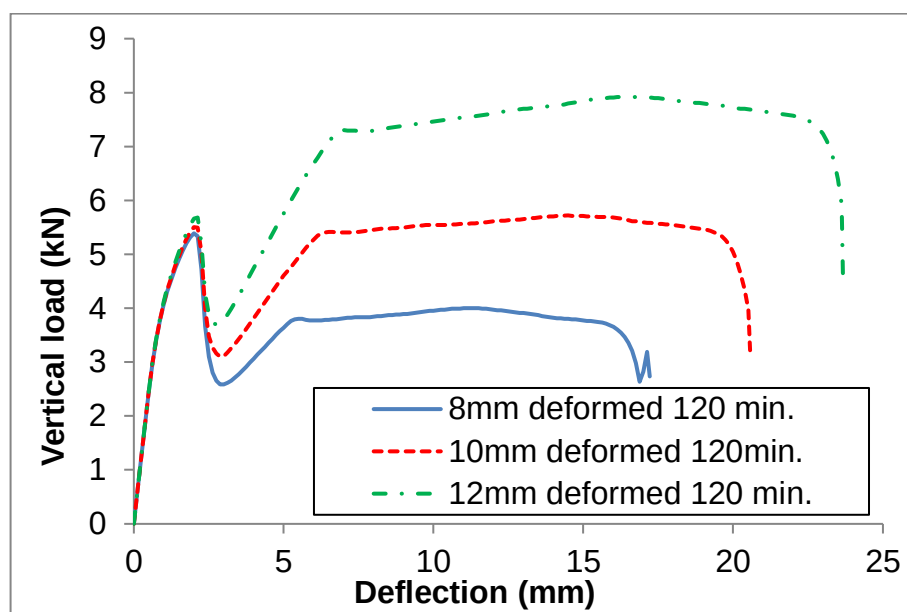


Figure 7.23. Load-deflection plot for deformed bars with different diameters after 120 minutes of fire duration

7.8 Discussion on the effect of bar diameter

Larger bar diameters have an increased ratio between cross sectional area and circumference and, therefore, an increased ratio between load carrying capacity and bond area. From the experimental research described in chapter 3, there are differences between the bond stress-slip response of bars with different diameter, but there is no clear trend between bar diameters and their retention of bond strength or stiffness at elevated temperature.

At ambient temperature, the bond slip was slightly increased with larger bar diameter, as shown in figure 7.18. The strain was also less concentrated for larger bar diameters at ultimate strain in the bar, as shown in figure 7.19. This is due to the increased ratio between load carrying capacity and bond area, which means that bond stress is higher, and therefore slip is higher, with the same amount of strain in the bar. The tolerable vertical deflection of the slab was greater for the bars with larger bar diameter than smaller bar diameters because of the increased slip which reduced the localisation of the strain of the reinforcing bar where it spans the crack. It is logical to expect that with larger diameter bars, such as 16mm or 20mm, the trend of increased bond slip and reduced strain concentration would continue given that the ratio between load carrying capacity and bond area continues to increase, but this has not been investigated.

After 120 minutes fire duration, the slip and strain responses, shown in figures 7.21 and 7.22, were very similar to those at ambient temperature, shown in figures 7.18 and 7.19. This is expected from this discussion in section 7.5 and it is due to the temperature at the bond being only around 53°C after 120 minutes fire duration. The load-deflection responses of specimens differed with different bar diameters at

elevated temperature in a similar way to the way in which they differed at ambient condition. This is because the difference between these specimens is in the response of the bar and the bond and neither of these will be significantly affected by the temperature difference between 20°C and 53°C. This demonstrates that the discussion on the effects of increased fire duration in section 7.5 is also valid for 10mm and 12mm diameter bars.

This demonstrates that crack spanning reinforcement with larger diameter will, with all else being equal, be less prone to rupture where large cracks develop around the perimeter of a slab bay during the formation of tensile membrane action because of the increased bond slip prior to rupture. This is the most important point of the discussion in this section and it is valid after different fire durations up to 120 minutes, and there is no obvious reason why it would not be true after longer fire durations.

7.9 Results and analysis - slab depths

The results presented in this section are for slabs of different depths. These results are considered at both ambient temperature and after 120 minutes fire duration. These simulations were numbered 1,3,8 and 9 in table 6.2. Comparison of these results demonstrates the effect which increased slab depth has on the behaviour of reinforcement at the locations of cracks around the perimeter of a slab bay during the development of tensile membrane action. These results are discussed in section 7.10.

The extent of localisation and concentration of strain in the bar at the point of failure was similar for 150mm and 200mm slab depths, as evident from figure 7.25. At ambient temperature, the bond slip was similar for 150mm and 200mm slab depths, as shown in figure 7.24. After 120 minutes fire duration, the slip was greater in the 150mm slab than the 200mm slab at the point of failure. The bond slip and axial strain in the 200mm slab was very similar at both ambient and elevated temperature, for the reasons discussed in section 7.5. The crack location was approximately 50mm further from the slab support in the case of the 150mm slab after 120 minutes fire duration. The crack location did not move significantly in any other scenario considered.

The vertical load carrying capacity of the slab decreased with reduced slab depth at ambient temperature and after 120 minutes fire duration, as shown in figure 7.26. At ambient temperature and after 120 minutes fire duration the 150mm slabs tolerated significantly greater vertical displacement than the deeper 200mm slabs before failure.

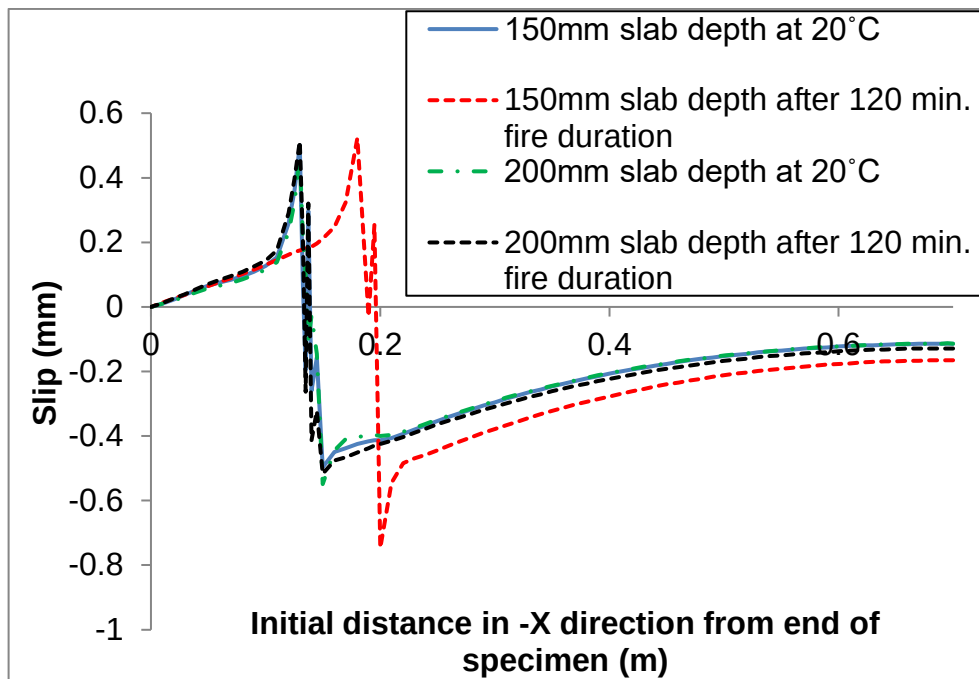


Figure 7.24. Plot of bond slip with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bars with slab depths of 150mm and 200mm at 20°C and after 120 minutes fire duration

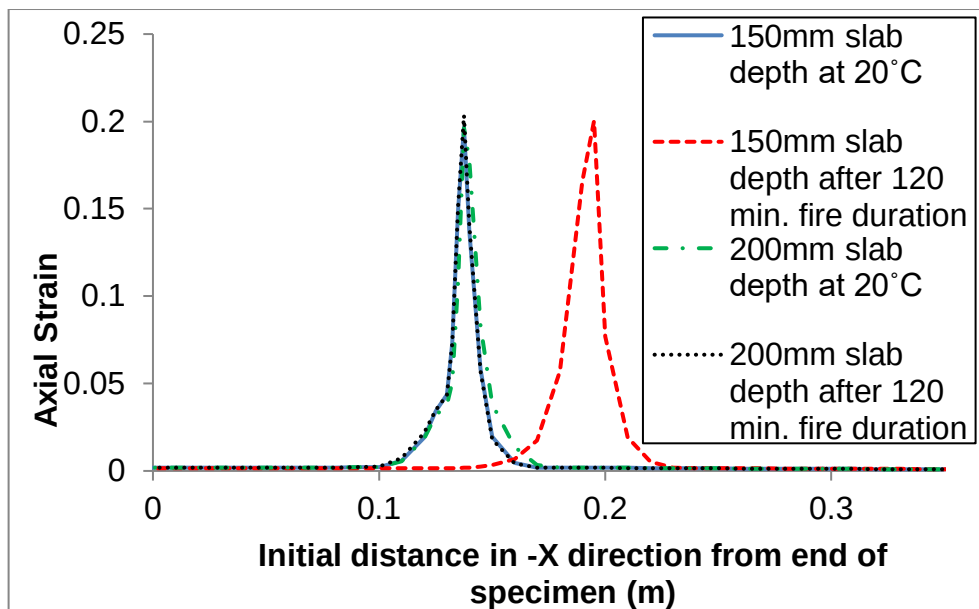


Figure 7.25. Plot of axial strain with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bars with slab depths of 150mm and 200mm at 20°C and after 120 minutes fire duration

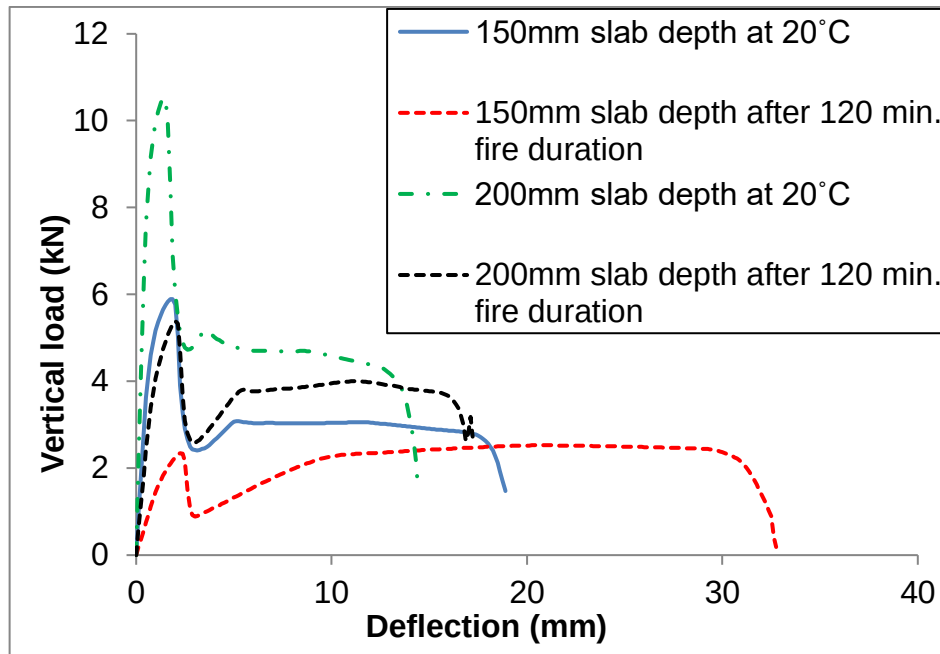


Figure 7.26. Load-deflection plot for 8mm diameter deformed bars with slab depths of 150mm and 200mm at 20°C and after 120 minutes fire duration

7.10 Discussion on the effect of slab depth

At ambient temperature, the bond slip and the axial strain in the bar at ultimate strain are similar for 150mm and 200mm deep slabs, as shown in figures 7.24 and 7.25. This is not surprising since there would be no change in the bond properties.

The tolerable vertical deflection of the 150mm specimen is greater than the 200mm specimen, as shown in figure 7.26. This is due to the point of crack rotation being closer to the reinforcing bar in the 150mm deep slab and, therefore, there can be more rotation at the crack to reach the same crack width. The increased rotation in the crack without rupture of the crack-spanning reinforcement facilitates more rotation of the slab near to the perimeter of the bay during the development of tensile membrane action. Therefore, based on the ambient temperature behaviour, shallower slabs will be more capable of withstanding the cracking which occurs

around the perimeter of the slab bay during the formation of tensile membrane action.

After 120 minutes fire duration, the bond slip was increased in the 150mm deep slab at the stage when ultimate strain was reached in the bar, relative to the ambient temperature condition, as shown in figure 7.24. The axial strain at the crack was also less concentrated at this stage in the 150mm deep slab after 120 minutes fire duration than at ambient temperature, as shown in figure 7.25. The bond slip and axial strain in the 200mm deep slab did not change significantly after 120 minutes fire duration relative to the ambient temperature condition. The change in bond slip and axial strain in the shallower slab can be explained by the temperature in the bond region reaching around 138°C rather than 53°C for the deeper slab. As the temperature in the bond region of the deeper slab was only 53°C for the deeper slab, the bond slip and axial strain are very similar to their ambient temperature values. At this temperature the bond stiffness was reduced by more than 20%, based on the input values. The result of the 150mm deep slab after 120 minutes of fire duration may, however, be affected by increased discretisation error as the crack formed in a coarser region of mesh in that single simulation.

The amount of vertical deflection which was withstood by the 150mm deep slab after 120 minutes fire duration was substantially greater than at ambient temperature, as shown in figure 7.26. This is due to a combined effect which included increased bond slip and reduced concentration of strain. A further contribution was from the reduced compressive strength of the slab in the lower region due to temperature. This reduction in compressive strength moved the flexural neutral axis higher, moving the point of rotation of the crack even closer to the bar than it was for the 150mm deep slab at ambient temperature. As the point of rotation of the crack was closer to the

bar, there can be even more rotation at the crack to reach the same crack width. Shallower slabs will be more capable of withstanding the cracking which occurs around the perimeter of the slab bay during the formation of tensile membrane action after 120 minutes fire duration than deeper slabs, and to an even greater extent than at ambient temperature.

Reduced slab depth means that the same rotation angle in the slab induces less strain in the reinforcement as it is closer to the point of rotation. This effect may be offset by the typically longer spans of deeper slabs as less rotation may be required for longer spans to transition from bending to tensile membrane action. However, the effect of slab span length has not been investigated in this research.

There may be a point at which a very shallow slab is too shallow to sufficiently insulate the reinforcement from weakening to the point of rupture due to high temperatures, but this was not the case in the slab depths considered here or in the 130mm deep Cardington test slabs.

The most important point of this section of the discussion is that shallower slabs are more capable of withstanding the cracking and slab rotation which occurs near the boundaries of a slab bay during the development of tensile membrane action than deeper slabs. After 120 minutes of fire, this effect is even more pronounced. Relative to the other parameters considered in this study, some of which are still to be discussed in later sections, slab depth is an important design consideration and it could prevent a slab from forming tensile membrane action without reinforcement rupture.

7.11 Results and analysis - additional loading

Where tensile membrane action forms in a slab which has lateral restraint at the boundaries, the tensile forces from the membrane action are carried by the reinforcement around the perimeter of the slab bay. A simulation was conducted with tension applied directly to the free end of the reinforcing bar. The intention of the simulation was to investigate the effect that the tension from the tensile membrane would have on the crack spanning reinforcement, and how it influences the bond slip and axial strain concentration in the bar at the crack. It is important to verify that the omission of the force from the tensile membrane action in the other simulations does not affect the behaviour which is being investigated.

The presence of an additional axial load did not significantly change the amount of bond slip near to the crack at the point of failure, as evident from figure 7.27. The presence of the tensile force did slightly reduce bond slip at one side of the crack, but the reduction was too small to be of significance. The distribution of bond slip was different in regions located away from the crack, towards the mid-span, which makes sense as the bar is being pulled out in this region by the applied tensile force. The localisation and concentration of strain in the bar at the crack location at the point of failure was very similar with and without additional axial load, as is evident from figure 7.28.

The additional axial load in the slab reduced the vertical load carrying capacity of the slab, as shown in figure 7.29. The presence of an additional axial load also reduced the amount of vertical deflection that could be tolerated by the slab before failure.

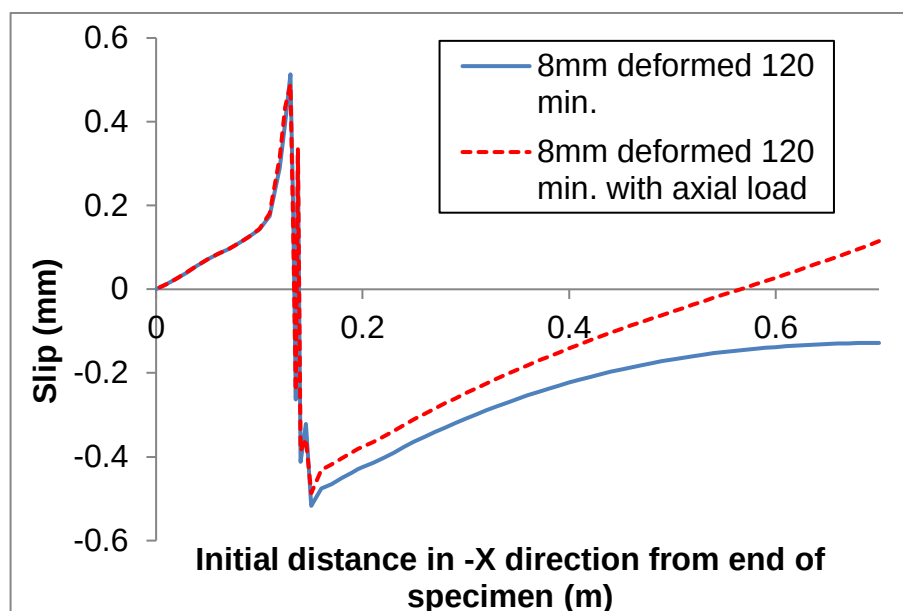


Figure 7.27. Plot of bond slip with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bars after 120 minutes of fire duration, under different loading conditions

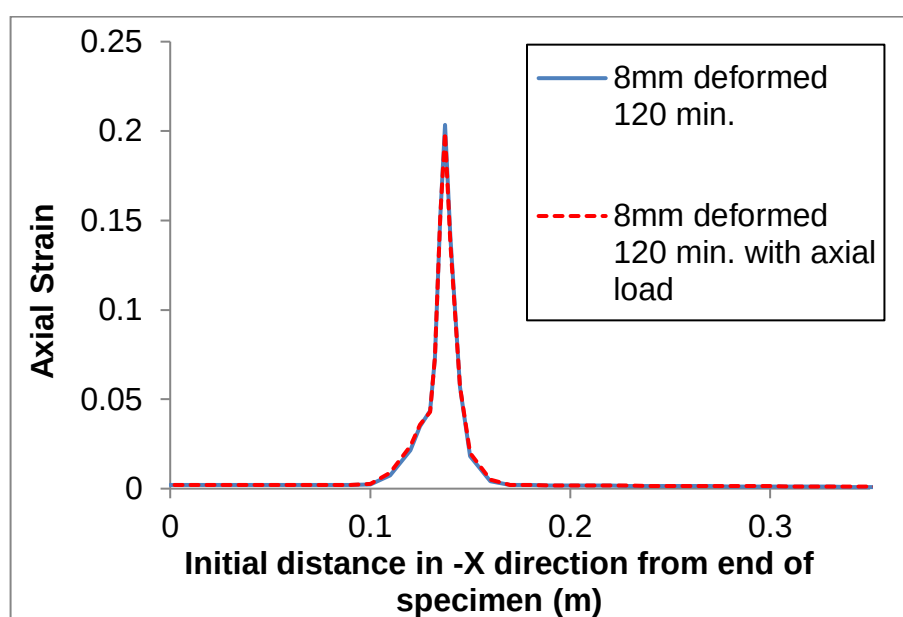


Figure 7.28. Plot of axial strain with respect to distance, measured along the reinforcing bar for 8mm diameter deformed bars after 120 minutes of fire duration, under different loading conditions

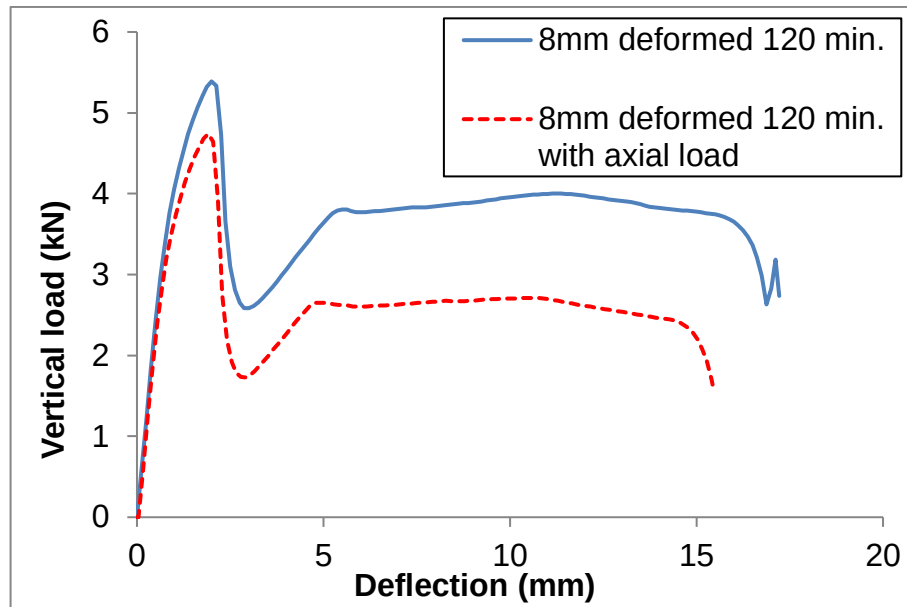


Figure 7.29. Load-deflection plot for 8mm diameter deformed bars after 120 minutes of fire duration, under different loading conditions

7.12 Discussion on the effect of additional load

Local to the cracking region, the amount of bond slip and the distribution of axial strain are not significantly affected by the presence of the additional load, as seen in figures 7.27 and 7.28. The bond slip direction did change further away from the crack, but the direction of slip reverted back along the bond length and did not significantly affect the bond slip behaviour at the crack. It is unsurprising that the vertical load carrying capacity of the specimen is reduced, as shown in figure 7.29, by the presence of additional loading and that the bar ruptures with less vertical deflection as the bar is subject to additional tensile stress.

The significant point from this discussion is that, although the stage at which rupture occurs is altered by the presence of this tension force, the bond slip and axial strain distribution at the point of rupture is not significantly affected. This establishes that it is reasonable for this tensile force to be omitted from the simulations as the results

are only used comparatively to study the effects of parameters on the bond slip, strain and rupture behaviour, and not to predict the actual stress-strain response of any specific slab.

7.13 Results and analysis - bond types at ambient temperature

This section presents the results of scenarios considering plain round bars, deformed bars and bars with perfect bond. The term perfect bond is taken here to refer to a bond interaction with no slip, but there is still a transfer of stress. Perfect bond is included in the comparison in order to assess the reasonableness of this as a design assumption. Comparing the results of these simulations is important to the current research in investigating the relative behaviour of these different bar and bond types at perimeter crack locations during the development of tensile membrane action. The perfectly bonded bars do undergo strain and, as with all simulations, the failure is considered to be the point at which the ultimate strain, $\varepsilon_{ae,\theta}$, in the bar is reached.

The simulations considered in this section are numbered 1,10 and 15 in table 6.2 and an extract of this table is included here, in table 7.1, to summarise the parameters under consideration. These results will be discussed in section 7.14. This section considers different bond types at ambient temperature in order to determine the ambient temperature behaviour before moving on to consider how elevated temperatures affect this behaviour. Sections 7.15 and 7.16 present results with different bond types at elevated temperatures.

Test	Bar Diameter (mm)	Bar surface/ bond type	Temperature	Ductility	Steel yield strength (N/mm ²)
1	8	Deformed	20°C	$\varepsilon_{au,\theta} = 0.15$ $\varepsilon_{ae,\theta} = 0.2$	500
10		Plain			250
15		Perfect			500

Table 7.1. Relevant parameters of the simulations presented in this section

At ambient temperature, at the point of rupture, the deformed bar had slipped more than the plain round bar, as shown in figure 7.30. It is important to note that this comparison was at the point of rupture of each bar and the specimen with deformed bar had undergone greater vertical deflection before it reached the point of bar rupture. The reasons for the increased slip of the deformed bar are explained in section 7.14, and further in section 7.19. The extent of localisation and concentration of strain was similar for both bar types, but was lower for the deformed bar, as shown in figure 7.31.

The slab with deformed bar reinforcement reached greater vertical deflection than other bar types. As evident from figure 7.32, the slab with plain round reinforcement had withstood less vertical deflection at failure than the perfect or deformed bar. The load capacity of the slabs was similar with different bar types until the bond had slipped sufficiently, at around 3mm vertical deflection, for the bars to be heavily utilised in terms of stress, as shown in figure 7.32. From this point, the capacity of the plain round bar slab was much lower due to the lower strength of the steel.

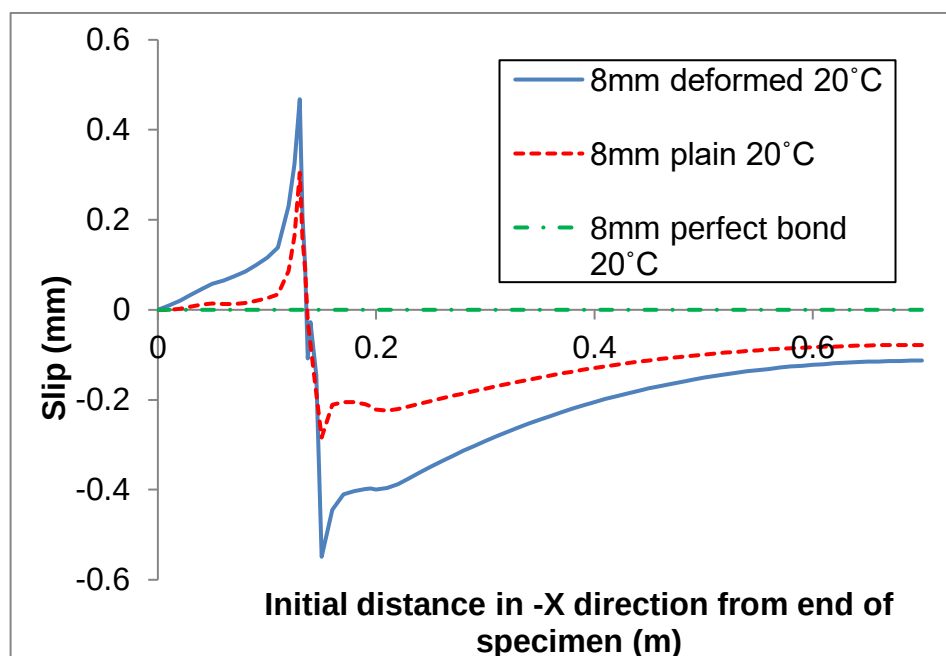


Figure 7.30. Plot of bond slip with respect to distance, measured along the reinforcing bar for 8mm diameter bar with different bar surfaces and bond types

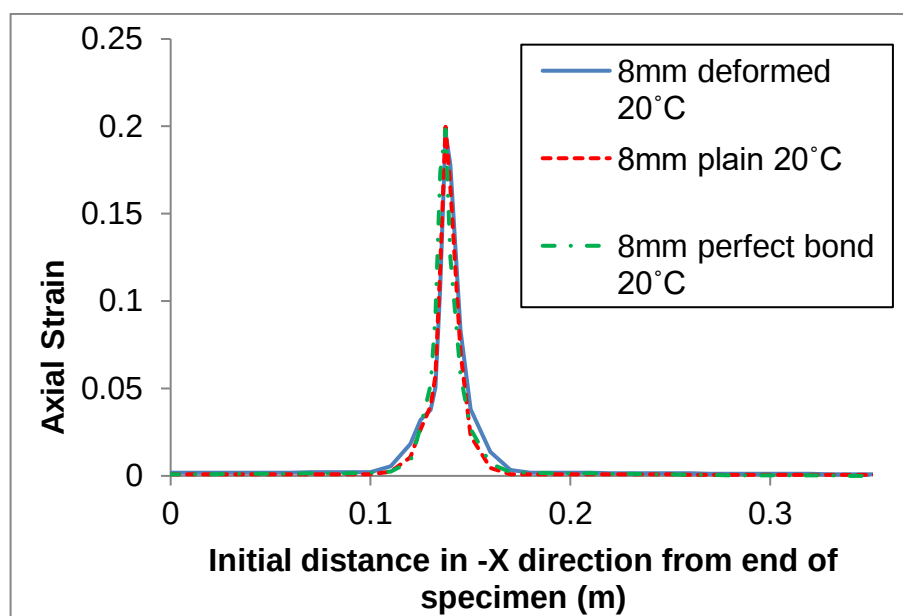


Figure 7.31. Plot of axial strain with respect to distance, measured along the reinforcing bar for 8mm diameter bar with different bar surfaces and bond types

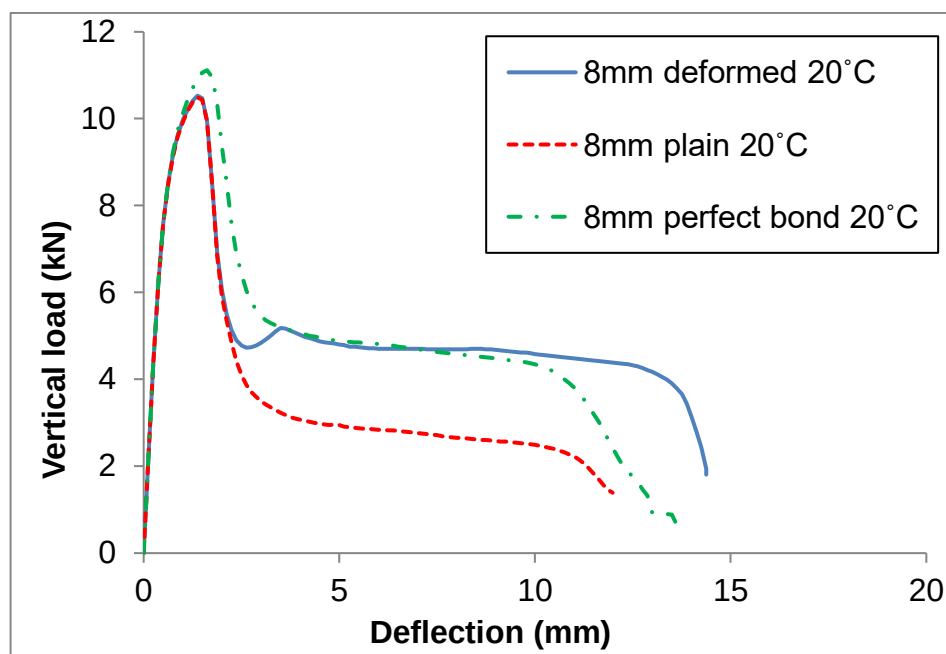


Figure 7.32. Load-deflection plot for 8mm diameter bar with different bar surfaces and bond types

In this section it is necessary to present an additional type of plot in order to explain the behaviour of the perfectly bonded specimen. These additional figures also aid in explanation of the behaviour of specimens with the plain round and deformed specimens. This explanation is part of the discussion in section 7.14. Figures 7.33 to 7.35 are plots of displacement in -X direction with respect to initial distance in the -X direction from the end of the specimen at the stage when ultimate strain is reached in the bar. Figures 7.33, 7.34 and 7.35 are for simulations 1, 10 and 15 respectively. The positions in the bar and in the concrete which are referred to in figures 7.33, 7.34 and 7.35 are shown in figure 7.36.

In each of figures 7.33, 7.34 and 7.35 there is an abrupt rise in displacement which occurs at the position of the simulated crack. As these are plots of displacement of points which are aligned with the X axis, a steeper gradient at any given point corresponds to higher axial strain at that point.

In figures 7.33, 7.34 and 7.35, the distribution of -X direction displacement is very similar at the bar edge and bar centre and, therefore, axial strain is similar at these two points in the bar. The difference in displacement between the bar edge and concrete edge in each of these figures is due to the bond slip at the interface. This refers to the same bond slip that is represented in figure 7.30. As expected, the distribution of -X displacement at the bar edge is the same as at the concrete edge for the perfectly bonded bar, as shown in figure 7.35, as there is no bond slip.

Comparing the distribution of displacement in the -X direction for the concrete edge and concrete interior, it is clear that there is a difference in displacement at these positions. This is true for all three bar types, as shown in figures 7.33, 7.34 and 7.35. This will be discussed in section 7.14

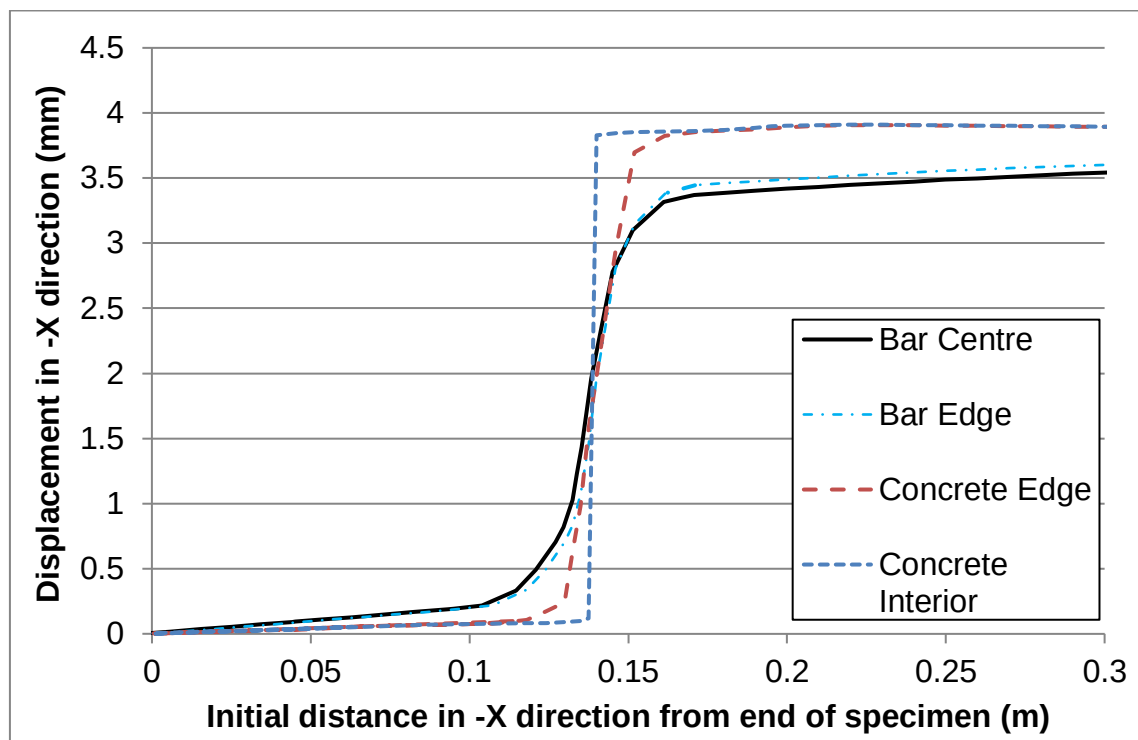


Figure 7.33. Plot of displacement in -X direction with respect to initial distance in the -X direction from the end of the specimen for 8mm deformed bar specimen at 20°C at the stage when ultimate strain is reached in the bar.

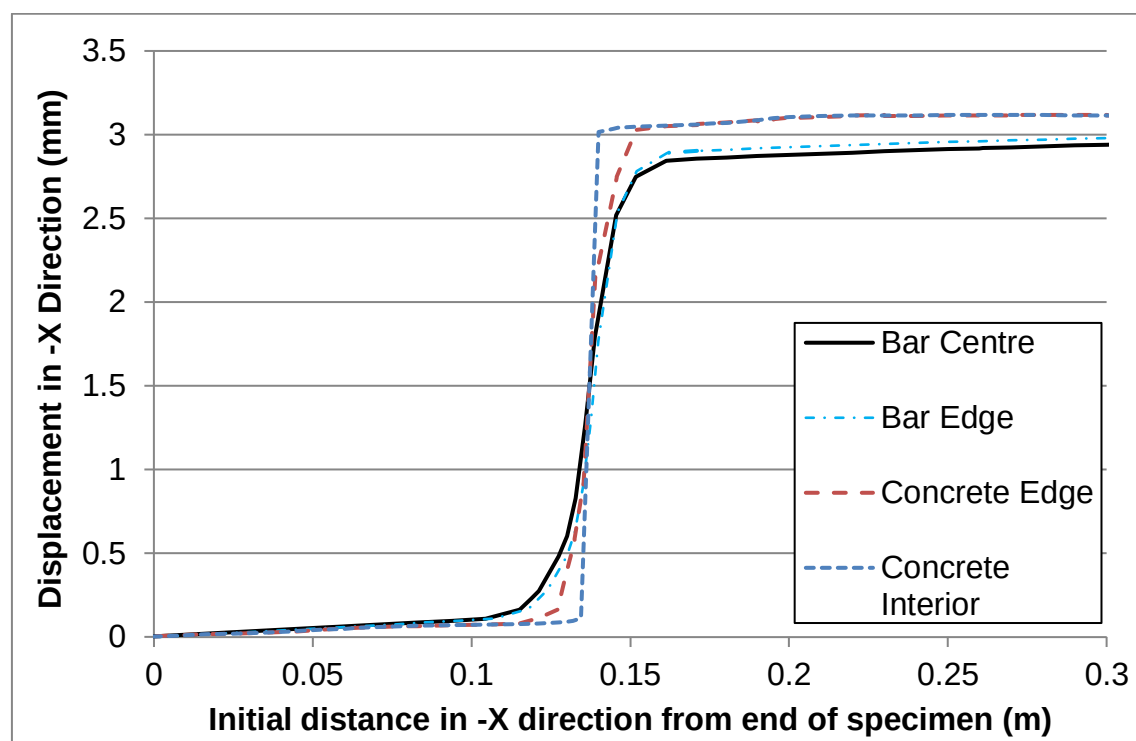


Figure 7.34. Plot of displacement in -X direction with respect to initial distance in the -X direction from the end of the specimen for 8mm plain round bar specimen at 20°C at the stage when ultimate strain is reached in the bar.

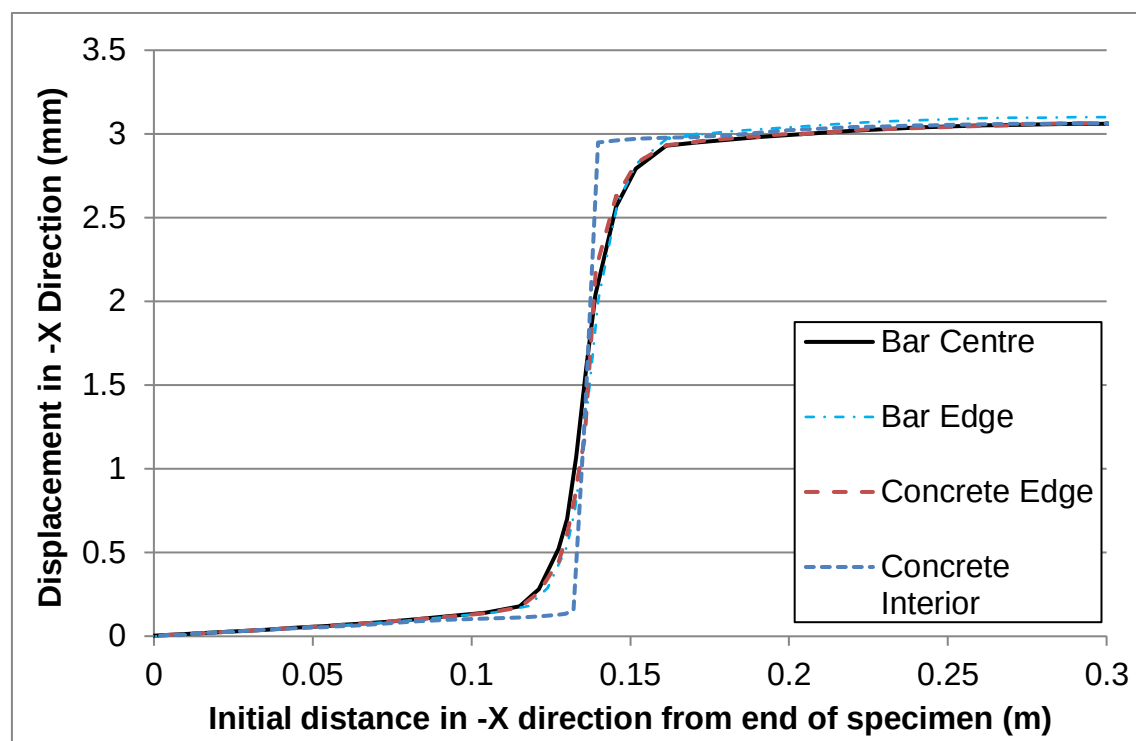


Figure 7.35. Plot of displacement in -X direction with respect to initial distance in the -X direction for 8mm perfectly bonded specimen at 20°C at the stage when ultimate strain is reached in the bar.

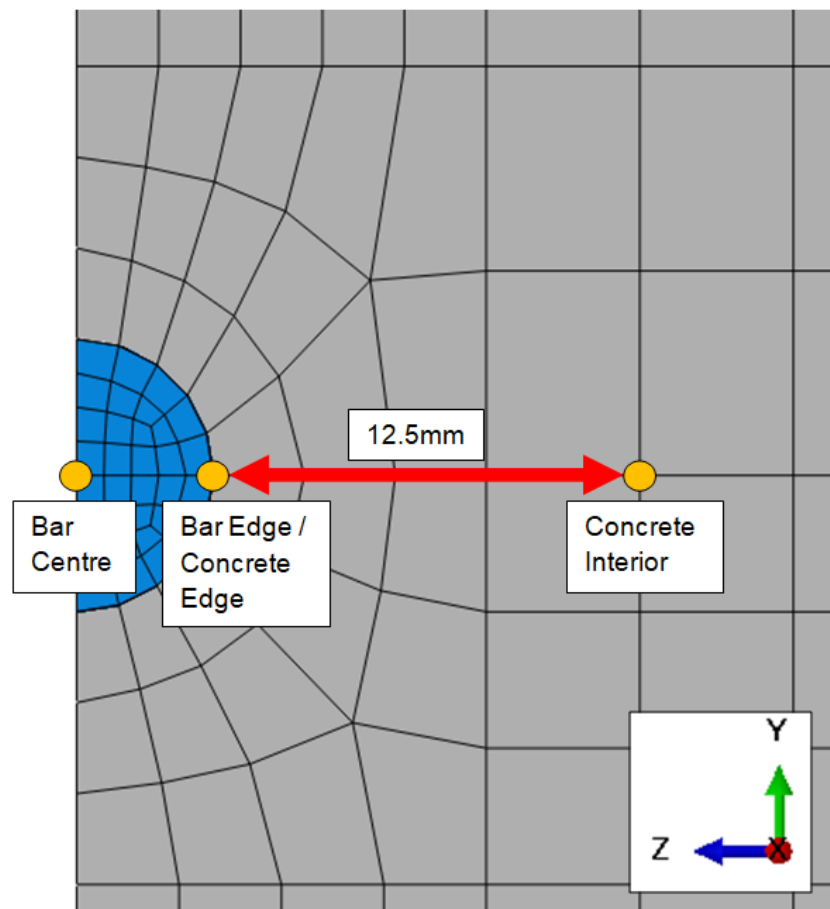


Figure 7.36. Annotated view of Abaqus model looking in -X direction at the region around the bar, showing the positions represented in figures 7.33 to 7.35 and 7.40 to

7.42

7.14 Discussion on the effect of bond types at ambient temperature

During the development of tensile membrane action, with horizontally restrained slabs, the slab rotates near to the boundary and cracking develops in these regions. The reinforcement which carries the tensile forces from the membrane action may be vulnerable to rupture due to concentration of strain in these regions. The effect of bar diameter, slab depth and temperature on the ability of a slab to withstand this slab rotation and cracking without reinforcement rupture have been considered in the previous sections. These parameters are all of interest as they are relevant considerations in structural fire design and, as has been explained in previous subsections of chapter 7, they can be significant factors.

The primary aim of this research is to investigate how modern deformed reinforcement will perform in terms of its ability to bridge the cracks which form around the boundaries of tensile membrane action. Design methods for tensile membrane action are, to a large extent, based on the observations of the Cardington fire tests, described in section 2.2, which used plain round bars, and it is not known from previous research if the different characteristics of deformed bars will make them more vulnerable to rupture. It is important to investigate this, in order to determine if these design methods are also suitable for modern deformed reinforcement. The discussion explains the relative performance of these bars at ambient temperature prior to investigating how these bars are affected by temperature in sections 7.15 and 7.16. Plain and deformed reinforcing bars are typically made of different steel grades which have significant differences in ductility, and this is considered in sections 7.18 and 7.19.

At ambient temperature, the deformed bar had more bond slip than the plain round bar at the stage of rupture, as shown in figure 7.30. The deformed bar had less concentrated strain at rupture than the plain round bar, as shown in figure 7.31. At 20°C, the initial bond stiffness of the deformed bar was around 20% higher than for the plain round bar, based on the parameter inputs of the model which are based on the calculated stress slip curves, as shown in figures D.1 to D.4, for the deformed bar and the experimental stress slip curves for the plain round bar in figure 3.26. The moderately higher stiffness of the bond response of the deformed bar will reduce the amount of bond slip at the point of rupture, yet the deformed bar still had greater bond slip.

The reason why the deformed bar had greater slip must be the higher strength of the material. Having higher steel strength means that more internal stress is required in the bar to reach the ultimate strain and rupture. The higher internal stress also results in a greater bond stress as more stress is transferred between the bar and the concrete either side of the crack. Increased bond stress results in an increase in bond slip, and as the material is almost twice as strong, including for strain hardening, the deformed bar has more slip overall, despite the roughly 20% stiffer bond response. It is clear that a large difference in material strength between reinforcing bars is substantially more significant than the modest difference in bond stiffness. It is important to note here that the typically higher ductility of plain bars is not considered in this comparison, and that this is investigated in section 7.18.

The specimen with the deformed bar withstood substantially more vertical deflection before the bar ruptured than the specimen with the plain round bar. This is due to the increased bond slip and less concentrated distribution of strain at the crack. In order to establish the relative performance of the deformed and plain round bars, the

greater ductility of the plain round bar must also be considered, and this is investigated in section 7.18.

The assumption of perfect bond between steel reinforcement and concrete is common in design models. As this means that there is no slip, and bond slip at a crack reduces the concentration of strain, it is logical to expect that this would be a conservative design assumption. It is useful to determine whether this assumption is appropriately, or excessively, conservative for design of hogging reinforcement of slabs where tensile membrane action is utilised in design.

Despite the lack of any bond slip, the perfectly bonded bar had slightly less concentration of strain than the plain round bar. This is due to the deformation of the concrete around the perfectly bonded bar, which is evident from figure 7.35. The concrete is deformed in the region around the bar, where it resists the bond stress. This deformation is also present around the plain round bar, but as the material of the perfectly bonded bar is almost twice as strong as the plain bar in this simulation, accounting for strain hardening, the bond stress around the perfectly bonded bar is larger at the stage of ultimate strain in the bar, and this greater bond stress results in greater deformation of the concrete surrounding the bar.

Relative to the deformed bar, which had the same material properties as the perfectly bonded bar, the bar with perfect bond had slightly greater concentration of strain at the crack, as shown in figure 7.31. This is expected as the deformed bar is able to slip, and therefore the concentration of strain is reduced. From figure 7.32, it is evident that the perfect bond assumption, although conservative, is not excessively conservative as the specimen with the perfect bonded bar still withstood a comparable level of vertical deflection to the other bars considered.

The most important point from this section of discussion is that, at ambient temperature and with equal ductility, a deformed bar is superior to a plain round bar at withstanding the cracking and deformation which occurs around the perimeter of a slab bay during the formation of tensile membrane action. This is due to greater steel strength which is typical of deformed bars being more significant than their modestly stiffer bond. Temperature and ductility are, however, important parameters to consider in this situation, and these will be considered in the following sections. The other important point from this discussion is that, at least at ambient temperature, the assumption of perfect bond is reasonable and appropriately conservative for design, although it would become more conservative if concrete deformation around the bar is not adequately accounted for.

7.15 Results and analysis - bond types after 120 minutes fire duration

fire duration

This section presents the results of simulations involving plain round bars, deformed bars and bars with perfect bond after 120 minutes fire duration. Comparison of these results is useful in investigating the relative behaviour of bars with these different characteristics at perimeter crack locations during the development of tensile membrane action in a fire scenario. The simulations considered in the section are those numbered 3,11 and 16 in table 6.2, and the relevant parameters to consider for these simulations are listed in table 7.2. It is important to note that the bars have the same ductility in these simulations. The results are discussed in section 7.14.

Test	Bar Diameter (mm)	Bar surface/ bond type	Fire Duration	Ductility	Steel yield strength (N/mm ²)
3	8	Deformed	120 minutes	$\varepsilon_{au,\theta} = 0.15$ $\varepsilon_{ae,\theta} = 0.2$	500
11		Plain			250
16		Perfect			500

Table 7.2. Relevant parameters of the simulations presented in this section

After 120 minutes of fire, the maximum local bond slip of the deformed bar was much greater than the plain round bar at failure, as can be seen from figure 7.37. The bond slip at failure was similar for each bar type to the ambient condition, which was presented in figure 7.30. The extent of localisation and concentration of strain of the deformed bar was slightly lower than that of plain round or perfect bonded bars, as shown in figure 7.38.

As at the ambient condition, the deformed bar had larger vertical deflection at failure than the other bar types, as shown in figure 7.39. The vertical deflection at failure was larger after 120 minutes of fire than in the ambient condition. The load capacity was larger after 120 minutes of fire than in the ambient condition. The load capacity of the slabs with plain round and deformed bars was similar until the bond had slipped sufficiently, at around 3mm vertical deflection, for the bars to be heavily utilised in terms of stress. With the bars fully utilised in terms of stress, the vertical load capacity was similar to the ambient condition.

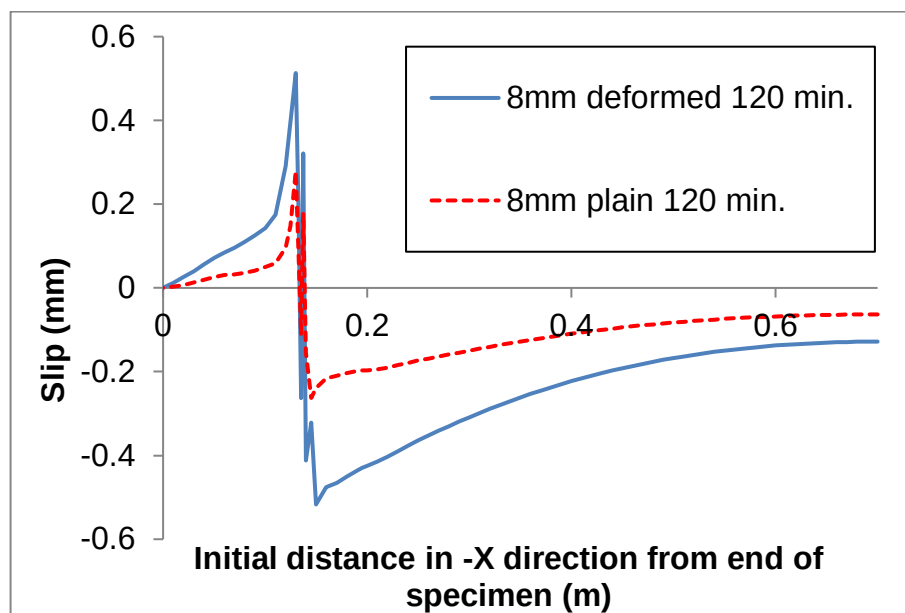


Figure 7.37. Plot of bond slip with respect to distance, measured along the reinforcing bar for 8mm diameter bar with different bar surfaces after 120 minutes of fire duration

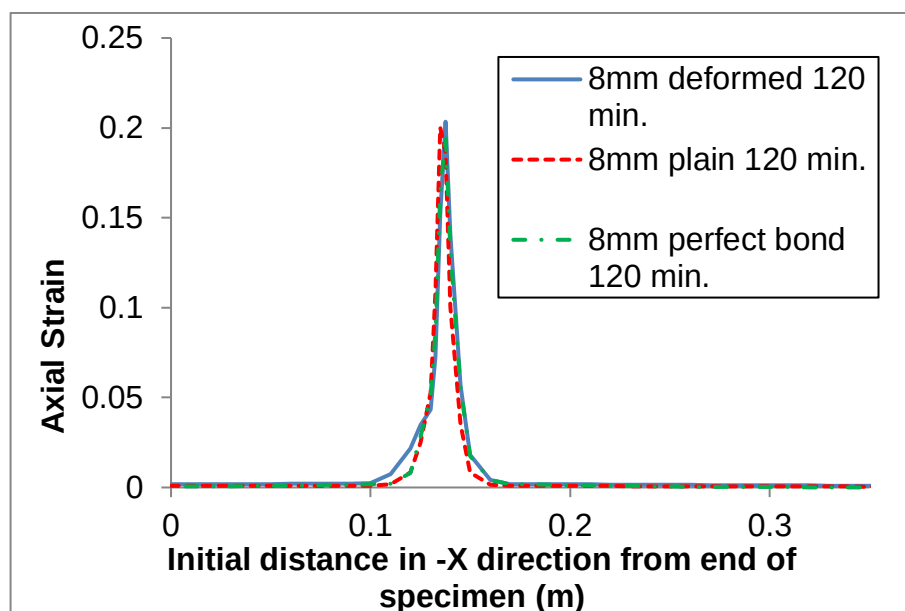


Figure 7.38. Plot of axial strain with respect to distance, measured along the reinforcing bar for 8mm diameter bar with different bar surfaces and bond types

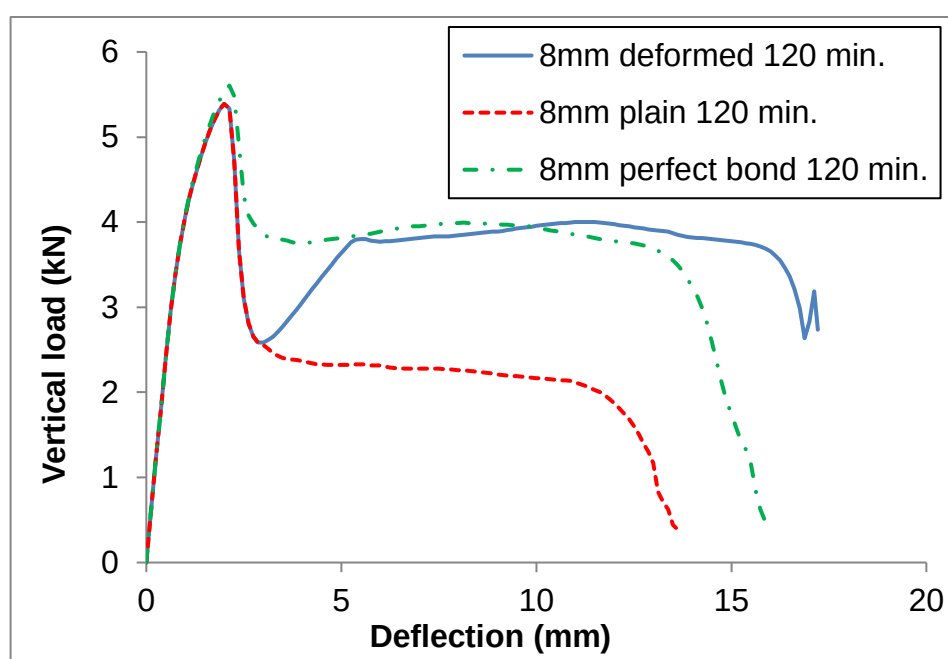


Figure 7.39. Load-deflection plot for 8mm diameter bar with different bar surfaces and bond types

Figures 7.40, 7.41 and 7.42 show the distribution of displacement in the -X direction with respect to initial position in the -X direction, after 120 minutes fire duration. The locations referred to in figures 7.40, 7.41 and 7.42 are shown in figure 7.36. The relevance of this type of figure was explained in sections 7.13 and 7.14. There was, as at ambient temperature, deformation of the concrete around the bar in all three simulations, as evident from the difference in -X direction displacement of the concrete interior and concrete edge locations. The results in figures 7.40, 7.41 and 7.42 are quite similar to those at ambient temperature. The main purpose of including these figures is to demonstrate that although the perfectly bonded bar did not have slip at the interface there was still concrete deformation around the bar and that this deformation was present for all three simulations.

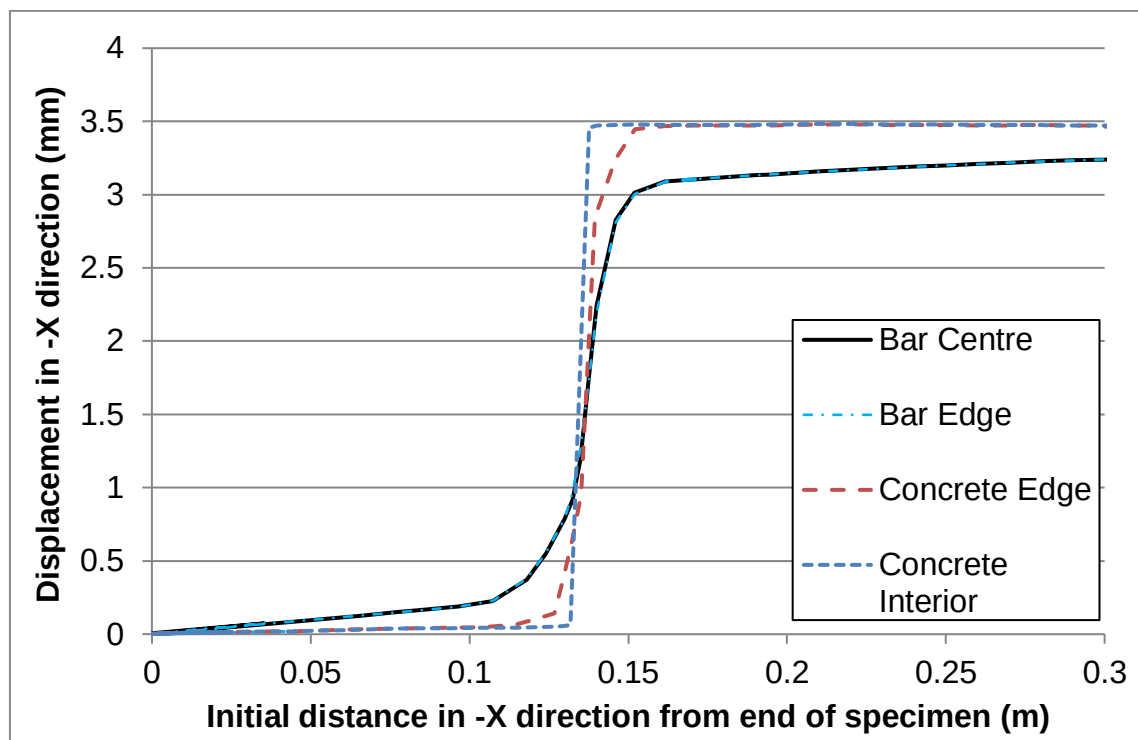


Figure 7.40. Plot of displacement in -X direction with respect to initial distance in the -X direction from the end of the specimen for 8mm deformed bar specimen at 120 minutes fire duration, at the stage when ultimate strain is reached in the bar.

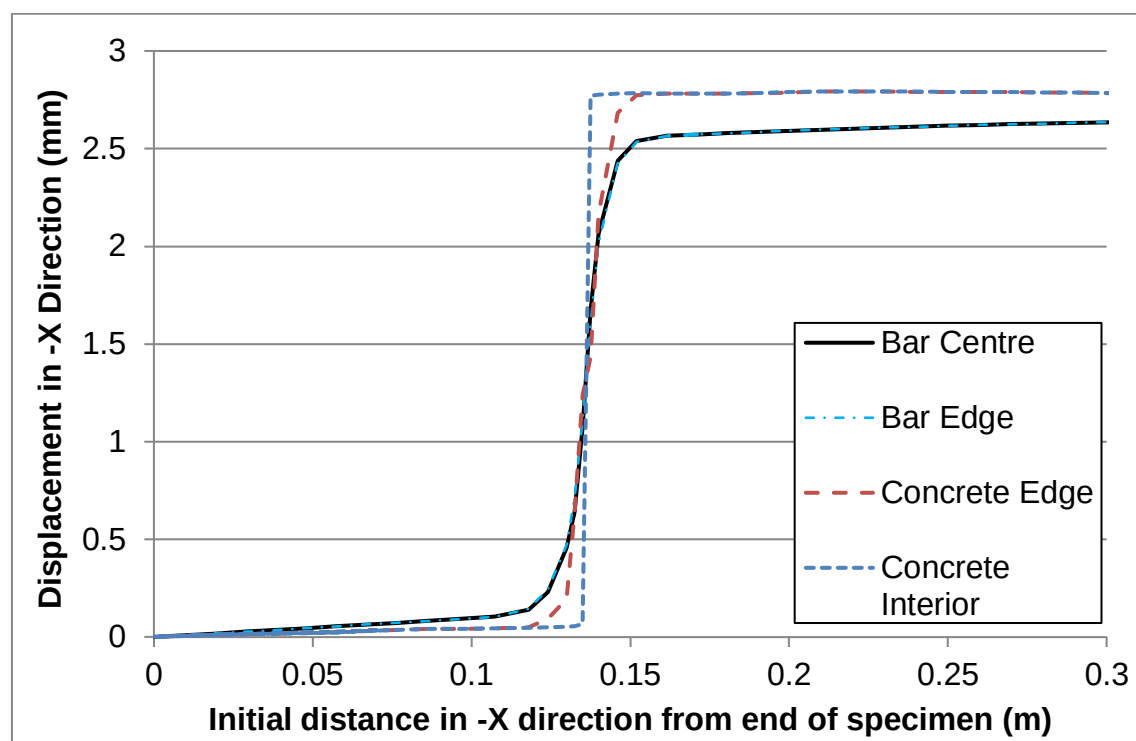


Figure 7.41. Plot of displacement in -X direction with respect to initial distance in the -X direction from the end of the specimen for 8mm plain round bar specimen at 120 minutes fire duration, at the stage when ultimate strain is reached in the bar.

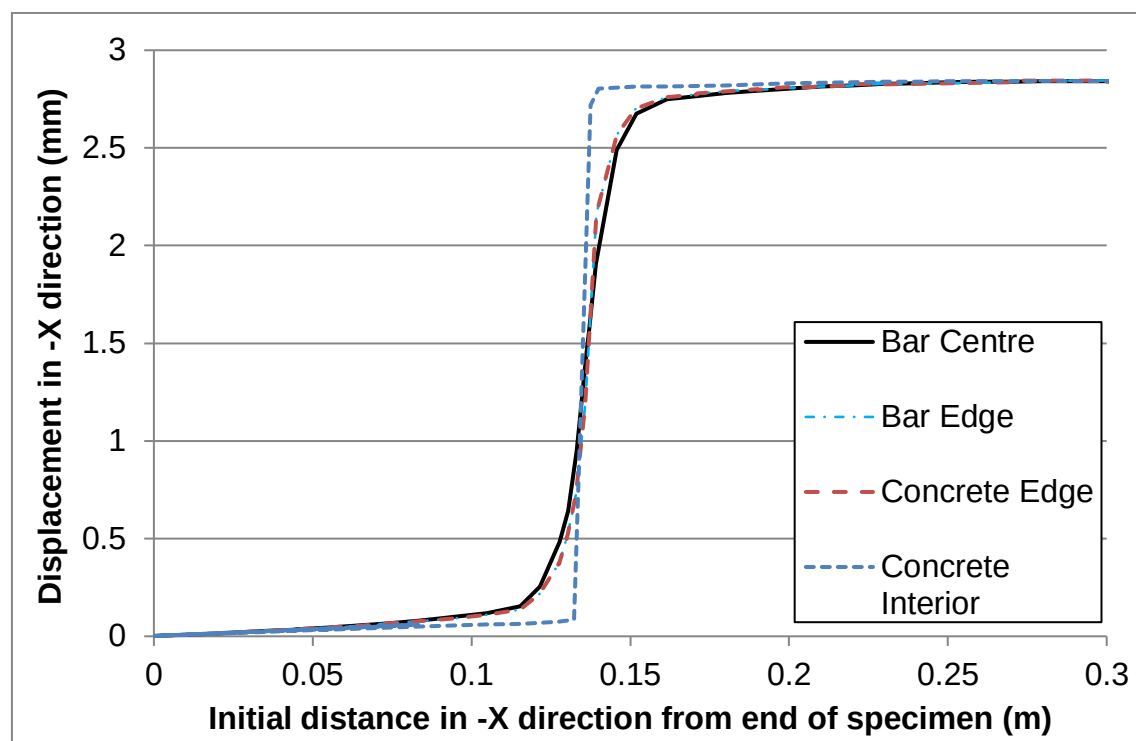


Figure 7.42. Plot of displacement in -X direction with respect to initial distance in the -X direction from the end of the specimen for 8mm perfectly bonded bar specimen at 120 minutes fire duration, at the stage when ultimate strain is reached in the bar.

7.16 Results and analysis - bond types at uniform elevated temperature

This section presents the results of further simulations involving plain round bars and deformed bars. These simulations were under uniform elevated temperature of 300°C. From comparison of these results, the effect of higher temperatures in the bonded region on the relative behaviour of plain round bars and deformed bars can be investigated. The simulations considered in the section are those numbered 5 and 12 in table 6.2, and the relevant parameters to consider here are listed in table 7.3. These results are discussed in section 7.17.

Test	Bar Diameter (mm)	Bar surface/ bond type	Temperature	Ductility	Steel yield strength (N/mm ²)
5	8	Deformed	300°C	$\varepsilon_{au,\theta} = 0.15$ $\varepsilon_{ae,\theta} = 0.2$	500
12		Plain			250

Table 7.3. Relevant parameters of the simulations presented in this section

At a uniform temperature of 300°C the plain round bar had slipped less than half as much as the deformed bar at the point of rupture, as shown in figure 7.43. The reasons for this are explained in section 7.17. The strain in the plain round bar was more localised and concentrated than in the deformed bar, as evident from figure 7.44.

After an almost identical peak in vertical load as the concrete tensile damage developed, the bond of the two bars slipped sufficiently that the bars became fully utilised. This is demonstrated in figure 7.45 by the point at which the vertical load stops reducing after the initial peak. The slab with the plain round bar had less

vertical load capacity than the deformed bar. The slab with the plain round bar sustained less vertical deflection than the deformed bar before failure.

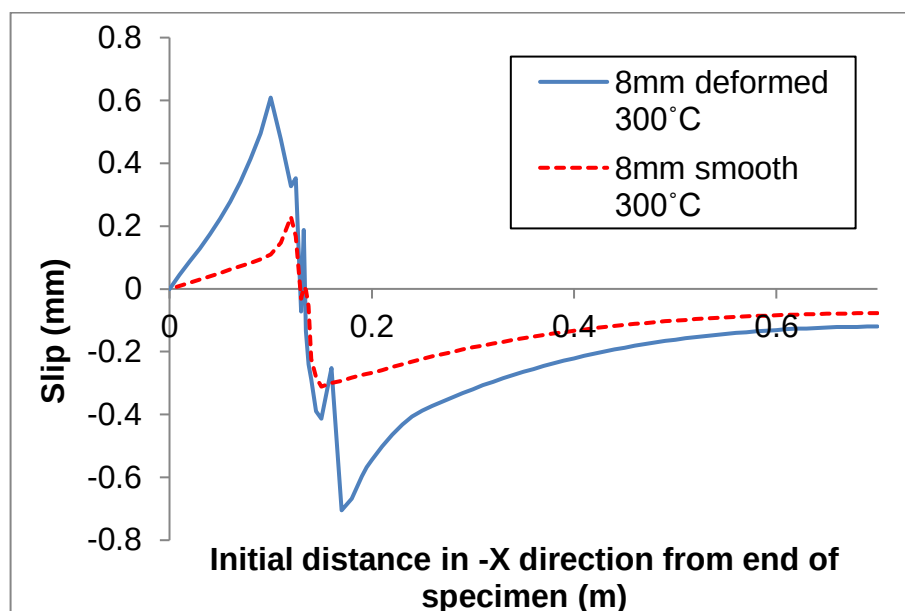


Figure 7.43. Plot of bond slip with respect to distance, measured along the bar for 8mm diameter bars of different surface types at 300°C uniform temperature

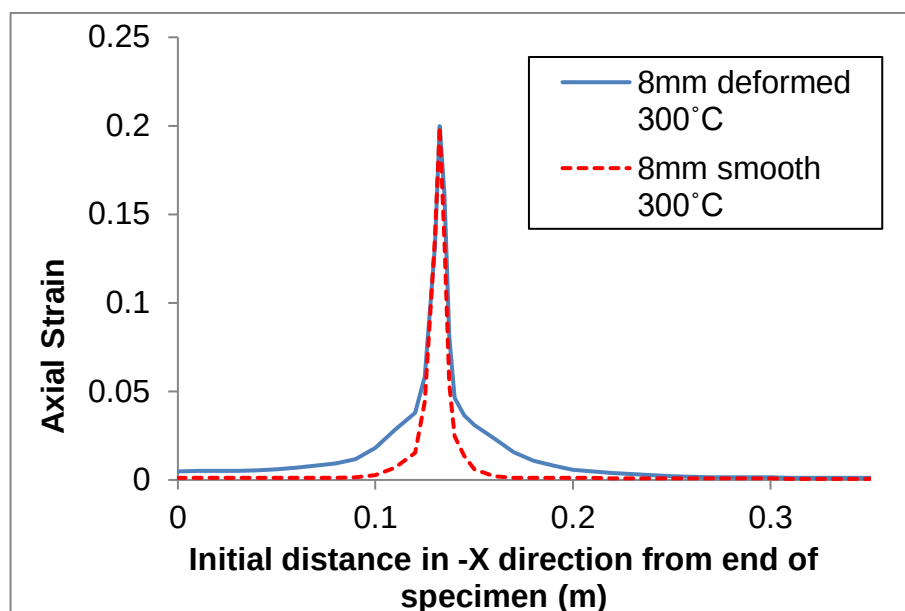


Figure 7.44. Plot of axial strain with respect to distance, measured along the reinforcing bar for 8mm diameter bars of different surface types at 300°C uniform temperature

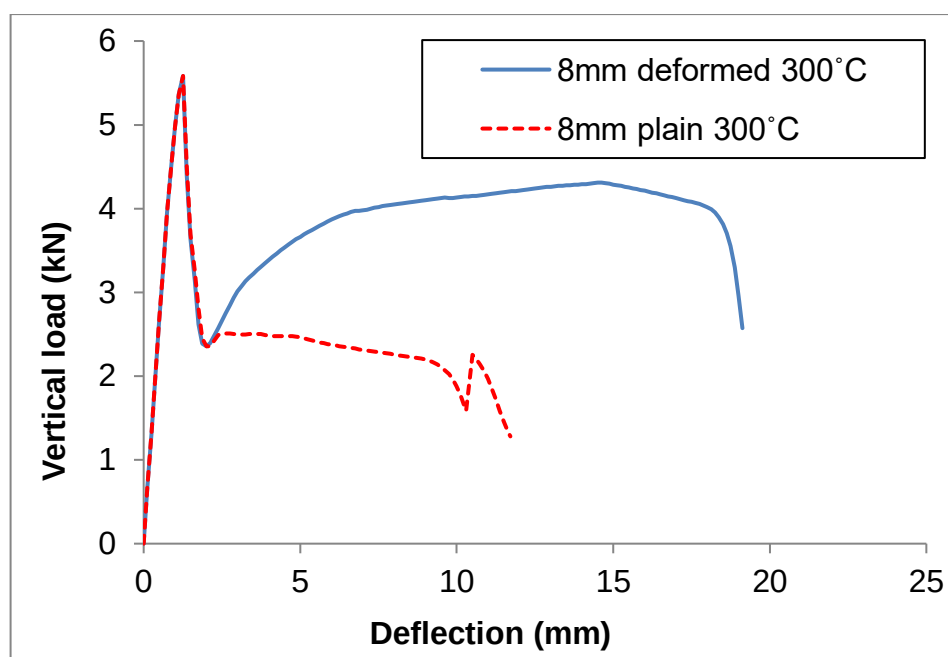


Figure 7.45. Load-deflection plot for 8mm diameter bars of different surface types at 300°C uniform temperature

7.17 Discussion on the behaviour of different bond types at elevated temperature

As at 20°C, the deformed bar had greater bond slip than the plain round bar after 120 minutes fire duration and at uniform elevated temperature of 300°C, as shown in figures 7.37 and 7.43. The deformed bar also had a less concentrated distribution of strain at the point of ultimate strain in the bar than the smooth bar at all temperatures considered, as shown in figures 7.38 and 7.44. The reasons for this are the same as at ambient temperature, with the greater strength of the deformed bar being more significant than the lower bond stiffness of the plain round bar.

The specimen reinforced with the deformed bar withstood greater vertical deflection than the plain round bar after 120 minutes of fire duration, as shown in figure 7.39. This is due to the greater bond slip at the stage of ultimate strain due to the higher strength of the deformed bar, as explained previously. This was also true at 300°C, but to an even greater extent, as shown in figure 7.45. The reason for this was the deformed bar had more bond slip and a much less concentrated distribution of strain at 300°C, due to reduced bond stiffness, than in the other temperatures considered, and this was not the case for the plain round bar, which had a more modest reduction in bond stiffness at this temperature.

After 120 minutes fire duration, the perfectly bonded bar had a similar distribution of strain at the stage where ultimate strain was reached to the plain bar, which did have lower strength, and the distribution was only modestly more concentrated than the deformed bar, which had the same strength. The amount of vertical deflection which could be withstood by the specimen with the perfectly bonded bar was more than the specimen with the plain bar and only approximately 20% less than the deformed bar

before reaching ultimate strain in the bar, as shown in figure 5.70. Therefore, as at ambient temperature, the assumption of perfect bond does produce conservative results after 120 minutes fire duration, relative to a deformed bar of similar strength, but not excessively conservative. As at ambient temperature, it can be seen from figure 7.42 that although there was no bond slip, there was deformation of the concrete around the bonded region.

The most important point from this section of the discussion is that, with equal ductility, deformed bars are more capable than plain bars of withstanding the cracking and rotation of the slab around the boundaries of the slab bay during the formation of tensile membrane action. This is true at both ambient temperature and elevated temperature. This is because the higher material strength which is typical of deformed bars is more significant than the difference in bond stiffness due to surface geometry in this scenario.

7.18 Results and analysis - mechanical properties of reinforcement

This section presents the results of simulations where bars with different mechanical properties have been modelled. The properties of ductility and strength have been considered. Load deflection plots alone are sufficient to demonstrate the significance of these parameters on the ability of the slab to undergo vertical displacement and, therefore, to withstand cracking and rotation of the slab near the support without reinforcement rupture. The simulations in this section are numbered 3, 7, 11, 13, 14 and 16 in table 6.2, and the parameters to be considered in this section are listed in table 7.4. The results in this section are discussed in section 7.19.

Test	Bar Diameter (mm)	Bar surface/ bond type	Fire duration	Ductility	Steel yield strength (N/mm ²)
3	8	Deformed	120 minutes	$\varepsilon_{au,\theta} = 0.15$	500
7				$\varepsilon_{ae,\theta} = 0.2$	
				$\varepsilon_{au,\theta} = 0.08$	
11		Plain		$\varepsilon_{ae,\theta} = 0.14$	250
				$\varepsilon_{au,\theta} = 0.15$	
13				$\varepsilon_{ae,\theta} = 0.2$	
				$\varepsilon_{au,\theta} = 0.18$	
14				$\varepsilon_{ae,\theta} = 0.23$	
	$\varepsilon_{au,\theta} = 0.15$				
16	Perfect	$\varepsilon_{ae,\theta} = 0.2$			

Table 7.4. Relevant parameters of the simulations presented in this section

The significance of ductility has been investigated because the previously presented results are all based on the Eurocode 4 Part 1-2 (BSI, 2005b) stress-strain curves and, therefore, ductility has been modelled as being equal for deformed and plain round bars in those simulations. The ductility of typical grade 500B deformed reinforcing bars and 250 grade plain round bars is, in reality, quite different. This is explained in section 6.4.

Strain value, $\varepsilon_{au,\theta}$, is the point at which stiffness damage onsets in the steel and the strain value $\varepsilon_{ae,\theta}$ is the strain at rupture. Strain values of $\varepsilon_{au,\theta}$ and $\varepsilon_{ae,\theta}$ of 0.18 and 0.23 respectively have been used for the plain bar in the scenario referred to as having 'increased ductility' in figure 7.46. The values of $\varepsilon_{au,\theta}$ and $\varepsilon_{ae,\theta}$ have been taken as 0.08 and 0.14 respectively for the deformed bar in the scenario referred to

as having 'reduced ductility' in figure 7.46. These values were based on experimental data by Du *et al.* (2015) and they are, therefore, expected to be more realistic than the values from BS EN 1994 Part 1-2 which were used in the other simulations, as explained in section 6.4.

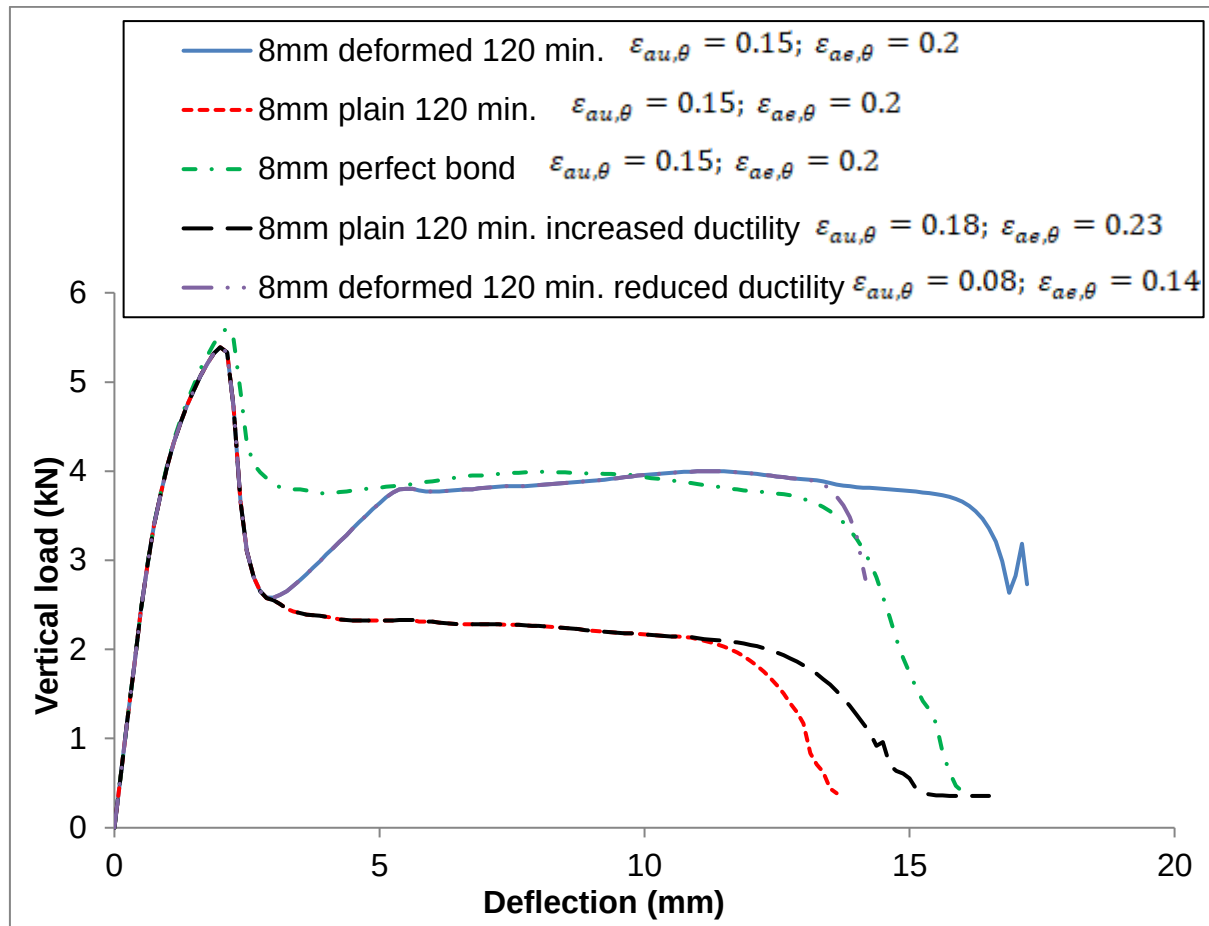


Figure 7.46. Load-deflection plot for 8mm diameter deformed bars after 120 minutes of fire duration, with different bar types and ductilities

For the specimen reinforced with a plain bar, increased ductility improved the ability of the specimen to withstand vertical displacement, as evident from figure 7.46. Reduced ductility of the deformed bar decreased the ability of the specimen to withstand vertical displacement. Even with these changes in ductility, the specimen reinforced with the deformed bar was still able to undergo slightly more vertical deflection than the one reinforced with the plain round bar before reaching the onset

of stiffness damage in the bar, when the load carrying capacity began to rapidly reduce.

A scenario in which a plain round bar had an increased strength has been considered. This an unrealistic scenario but it is worthwhile because it allows the separate effects of bar shape and steel strength to be compared in isolation. In the scenario labelled '500MPa' in figure 7.47 the specimen is reinforced with a plain round bar with 500MPa yield strength and the same strain hardening characteristics as was assigned to the grade 500B deformed bars.

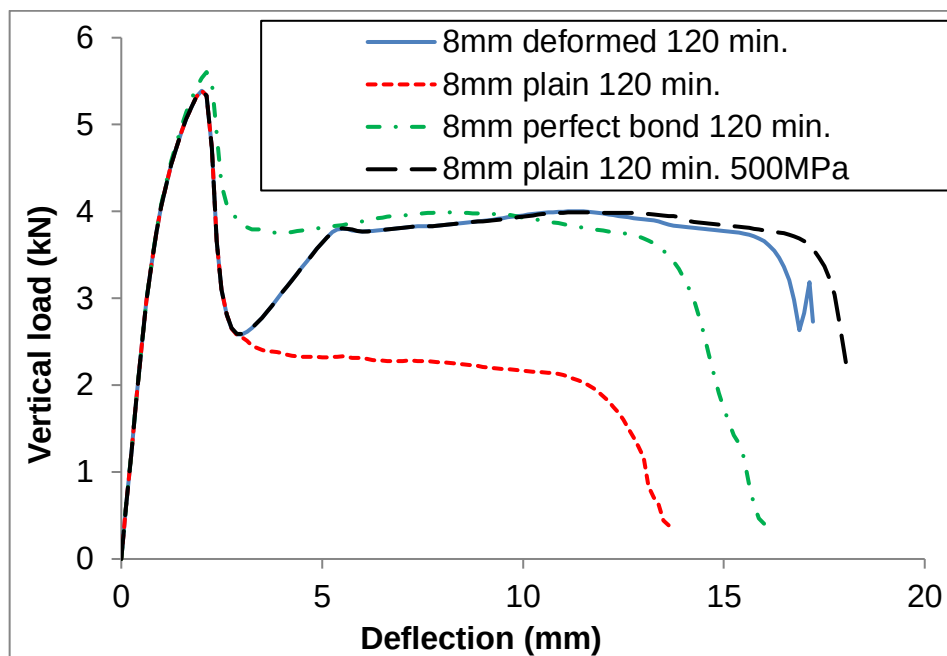


Figure 7.47. Load-deflection plot for 8mm diameter deformed bars after 120 minutes of fire duration, with different bar types and strengths

The specimen which was reinforced with a stronger plain round bar was able to undergo substantially more vertical deflection than the standard plain round bar before reaching the critical point where the load carrying capacity began to rapidly reduce due to the onset of stiffness damage in the reinforcing bar. With equal material properties, the specimen reinforced with the plain round bar performs

similarly to the deformed bar, but has slightly greater ability to withstand vertical deflection before damage onset in the bar.

7.19 Discussion on the performance of reinforcing bar types

The discussion in sections 7.14 and 7.17 has considered the effects of bar type on the crack-spanning behaviour of reinforcement as cracking and slab rotation occurs around the boundary of a slab bay during the formation of tensile membrane action. In sections 7.14 and 7.17, this discussion was limited to the effects of bond stiffness, due to surface geometry, and the effects of material strength. It was found between sections 7.14 and 7.17 that the increased material strength which is typical of deformed bars is more significant in the scenario being considered than the difference in surface geometry between deformed and plain round bars. Plain round bars are typically made from more ductile steel than deformed bars, and it is important that this difference is also considered. This section considers the relative performance of different reinforcing bars, considering the effects of steel ductility, strength and surface geometry.

7.19.1 Effect of ductility

Figure 7.46 demonstrates the effects of ductility. The results denoted as 'increased ductility' and 'reduced ductility' are for specimens where more realistic values from experimental results, rather than Eurocode 4 Part 1-2 (BSI, 2005b), have been used to define strain inputs, including for the critical value of $\varepsilon_{au,\theta}$, at which point the steel begins to weaken. It is apparent that increasing ductility of plain round bar to a more realistic value has modestly increased the ability of that specimen to withstand vertical deflection. The reduced ductility of the deformed bar has had the opposite

effect on that specimen. The effect of ductility is sufficiently large that there can be a high degree of confidence in these general observations, even if the level of uncertainty in the inputs and assumptions of the model prevent the magnitude of the effect from being reliably quantified with a high degree of accuracy.

7.19.2 Effect of surface geometry

Comparing the results of deformed bar with the hypothetical 500MPa plain round bar in figure 7.47 shows the effect of the bond properties due to surface geometry alone, as the mechanical properties are equal in this case. It is evident that in this case, the specimen reinforced with the plain round bar was able to withstand marginally greater vertical displacement than the deformed bar specimen. This agrees with the discussion in sections 7.14 and 7.17 that the lower bond stiffness of the plain round bar is an advantage, but demonstrates that the benefit is only marginal with such low levels of bond slip. The level of confidence in this particular observation is limited by the small number of tests which the bond stiffness inputs were based on, as this is highly sensitive to these input values.

The ribs of deformed bars cause wedging action in the surrounding concrete generating normal stresses in the concrete. These effects are present in the model for deformed bars through the interaction definition described in section 5.4, and the damage of the concrete is represented using the concrete damage plasticity model within Abaqus, as described in sections 2.7 and 5.3. There was no obvious effect from wedging action in the results, but it was not examined in isolation. The normal stresses from wedging action are present in the results of all of the simulations with deformed bar and, therefore, these effects are considered in the comparisons between bar types.

7.19.3 Effect of steel strength

Comparison of the relative performance of the standard plain bar and the 500MPa plain bar in figure 7.47 allows us to see the effect of material strength in isolation, as all other factors are identical between these specimens. It is evident that increased material strength of the reinforcement greatly increased the ability of the specimen to withstand vertical deflection. Increased steel strength is, therefore, very beneficial in allowing a bar to facilitate slab rotation and cracking near the perimeter without reinforcement rupture. The performance of the 500MPa plain bar in figure 7.47 suggests that this would be an effective reinforcement in this scenario, although this type of bar is not typically available as a reinforcing material.

It is apparent from comparing the performance of these specimens with the deformed specimen in figure 7.47 that the effect of material strength is much more significant than bar surface geometry. Despite the level of uncertainty from the many inputs and assumptions of the model, the effect of increased material strength in the results is sufficiently large that there can be a high degree of confidence that increased bar strength is significantly beneficial for the situation being considered.

7.20 Overall performance of deformed bars relative to plain round bars

Deformed bars typically have higher material strength than plain round bars and this helps them in bridging cracks around the perimeter of a slab bay during the formation of tensile membrane action. Plain round bars typically have greater ductility than deformed bars and this helps them in bridging cracks in the same situation. The effects of surface geometry are not as pronounced, and although the

plain round bars were found to have lower bond stiffness than deformed bars at the specific temperatures considered, the difference is marginal.

Comparing the results of the deformed bar with 'reduced ductility' and the plain bar with 'increased ductility' in figure 7.47, we can see that the deformed bar is slightly superior overall in its ability to withstand slab rotation without reinforcement rupture in the scenario considered. This comparison takes into account the differences in ductility, material strength and surface geometry. The benefit of the higher strength of the deformed bar, therefore, marginally outweighs the effects of ductility and surface geometry which were observed to benefit the plain round bar in these simulations.

Given the level of uncertainty and limited range of test data, the complexities of the model and the similarity of these results in terms of vertical deflection there is not sufficient basis to claim that deformed bars are superior to plain round bars in allowing a slab to develop tensile membrane action. It does appear, however, that deformed bars can be expected to perform similarly well to plain round bars in facilitating the rotation and cracking in a slab during the development of tensile membrane action. In general, the observations demonstrate that the tensile membrane design approach, which has been developed on the basis of the Cardington tests with plain round bar, should also be applicable to deformed bars.

7.21 Design Implications for Reinforced Concrete and Composite Slabs

Composite slab design methods, such as the method described in the Steel Construction Institute's 'Fire Safe Design' guide (Newman *et al.*, 2006), which utilise tensile membrane action in fire scenarios are based on the observations of the

Cardington tests where plain round bar was used. It is not known from previous research whether the superior bond performance of deformed bars may make them more prone to rupture as a slab cracks and rotates around the perimeter of the slab bay. It is important to determine whether tensile membrane action can be fully utilised in design of horizontally restrained slabs reinforced with modern deformed reinforcement.

Deformed bars typically have greater steel strength than plain round bars and this is beneficial in spanning cracks around the perimeter of a slab bay during the formation of tensile membrane action. Plain round bars typically have greater ductility than deformed bars and this helps in the same situation. Provided the bond does not slip beyond the point of peak stress, bond stiffness resulting from each bar type is quite similar and this is not as significant as the material properties of the bar.

The research findings indicate that deformed reinforcing bars should perform similarly well to plain round bars, due to their greater strength compensating for their lower ductility and marginally stiffer bond. It is therefore, reasonable to utilise tensile membrane action in structural design of composite slabs reinforced with deformed reinforcement in fire scenarios.

Slab depth is a highly important factor in determining the capability of a slab to transition into tensile membrane action. The observation that tensile membrane action developed in a 130mm deep composite slab in the Cardington tests does not establish that tensile membrane action could form in a deeper slab without reinforcement rupture. The ability of a slab to fully transition into tensile membrane action depends on the ability of the slab to rotate around the perimeter of the bay without crack-spanning reinforcement rupturing. In deeper slabs, the same rotation

angle requires a larger crack width at the position of reinforcement near to top of the slab. For this reason, with all else being equal, reinforcement will be more prone to rupture in deeper slabs than shallower slabs around the perimeter of a slab bay during the development of tensile membrane action.

Regardless of bar type, bar diameter or slab depth, it is critical that any design which utilises tensile membrane action accounts for the strain localisation in the hogging reinforcement at crack locations. As current design models will not, in general, be capable of modelling this accurately a conservative approach is appropriate. It has been demonstrated that modelling perfect bond of the hogging reinforcement is appropriately, and not excessively, conservative when considering reinforcement rupture in designs utilising tensile membrane action in fire scenarios. This research has established, therefore, that it is reasonable for designers to use this assumption of perfect bond for the design of hogging reinforcement of a slab with tensile membrane action development.

The FE model developed in this research was created for the purpose of the current parametric study and is not intended for direct application in design in its current form. The modelling approach is a suitable foundation for the development of further research models for studying bond and reinforcement under various conditions.

The results which have been considered are limited to modelling hogging reinforcement under conditions in which it will stay relatively cool. The bond slip did not reach the point corresponding to peak bond stress, and there may be circumstances where this could occur. In a situation where bond slip went beyond the peak bond stress then the response would be very different to those observed in

the simulations, but this has not been investigated. This may occur in slabs with larger bar diameters, higher temperatures or different concrete grades.

8 Conclusions and further work

Existing structural design methods for concrete floor slabs in fire conditions take advantage of the enhanced load bearing capacity provided by tensile membrane action. For slabs to fully transition into tensile membrane action, large cracks develop around the boundaries of the slab bay and it is critical to the integrity of the slab that the reinforcement is able to bridge these cracks without rupture. The aim of this research was to determine whether modern deformed reinforcement will be more prone to rupture than plain round reinforcement at hogging cracks which form around the perimeter of a slab during the development of tensile membrane action in a fire scenario.

In order to meet this research aim it was necessary to characterise the elevated temperature bond behaviour of modern deformed bars and plain round bars as this allowed the relative performance of these bar types to be investigated and the suitability of design methods based on plain round bars for design of slabs reinforced with modern deformed bars to be assessed. The aims of this research have been achieved by conducting elevated temperature pull-out experiments to characterise bond at elevated temperatures and by conducting numerical modelling research to study the behaviour of crack-spanning reinforcement during the development of tensile membrane action in a fire scenario.

8.1 Conclusions regarding reinforcement bond at elevated temperature

The bond response of modern deformed reinforcement at elevated temperatures has been investigated as it is not fully understood from previous research. The effect of elevated temperatures on bond stiffness, in particular, is not known from previous research and this has been investigated. This information was required in order to investigate the crack-spanning behaviour of modern deformed reinforcement during the development of tensile membrane action, and to determine whether existing tensile membrane action design methods based on observations of plain round bars are appropriate for modern deformed reinforcement.

The bond behaviour of deformed bars and plain round bars at elevated temperatures has been characterised. Pull-out experiments were performed on 8mm diameter plain round bars and 8,10 and 12mm diameter deformed bars at temperatures up to 570°C. The results of these experiments have been presented in the form of bond stress-slip curves. The conclusions of the elevated temperature bond investigation are as follows:

- The bond strength of deformed bars generally decreased with increasing temperature, but the trend followed the down-up-down trend which has previously been observed in concrete compressive strength. The decline in bond strength with increasing temperature was more rapid than the previously measured decline in concrete compressive strength.
- The bond strength of deformed bars decreased at 110°C and 210°C relative to the ambient temperature condition. Bond strength of deformed bars was greater at 290°C than at 110°C and 210°C, but was significantly lower at

570°C than at any other temperature considered. The down-up-down trend in bond strength of deformed bars is due to the similar trend in concrete compressive strength because the bond strength of deformed bars is primarily due to mechanical interlock, which depends on concrete strength to resist bearing stresses of the ribs.

- The bond strength of plain round bars increased at 110°C and 210°C relative to the ambient temperature condition. This is due to differential thermal expansion and could only be observed when specimens are tested hot rather than after cooling. At 570°C the bond strength of plain round bars had almost disappeared due to differential thermal expansion and the deterioration of the cement paste at this temperature.
- The proportion of bond strength retained by plain bars was greater than deformed bars at elevated temperatures below 290°C. At around 290°C, both bar types retained a similar proportion of bond strength, but at 570°C plain bars performed much worse than deformed bars in terms of bond strength retention.
- There was no discernible relationship between bar diameter and bond strength retention at elevated temperature.
- The stiffness of the bond stress-slip response generally decreased with increasing temperature.
- The initial stiffness of the bond stress-slip response was quite similar for plain bars and deformed bars of different diameters at each temperature considered, except at 570°C where the initial stress-slip response of the plain bar was much less stiff.

A local bond model applicable at elevated temperatures has been developed by modifying the existing fib local bond model (fib, 2013). This modified model includes a bond strength reduction factor which is a function of the temperature dependency of concrete compressive strength. The quality of fit of the bond strength reduction factor with current and previous experimental results has been demonstrated. Previous local bond models are not specifically intended for elevated temperature use so this local bond model will be useful for design and research relating to reinforcement bond and anchorage at elevated temperature.

8.2 Conclusions regarding reinforcement behaviour In reinforced concrete and composite slabs In fire scenarios

The following conclusions refer to a scenario where tensile membrane action forms in a composite or reinforced concrete slab with continuity over perimeter supporting beams during a fire scenario. The performance of modern deformed and plain round reinforcing bars in this scenario has been investigated through numerical modelling in order to meet the research aim. The numerical modelling approach was developed for this research and it incorporates flexural crack development, steel rupture and the bond interaction, which was characterised from the pull-out experiments, at elevated temperatures.

In order to meet the further objectives of this research, the effects of different parameters on the ability of the slab to withstand rotation and cracking without reinforcement rupture during the development of tensile membrane action have been investigated. The parameters which have been considered are temperature, fire

durations, slab depth, bar diameter and bar type. The conclusions from this numerical modelling research are as follows:

- Composite slab design methods utilising tensile membrane action do not need to be changed for use with deformed reinforcing bars in place of older plain round reinforcing bars, within the range of parameters considered in this research.
- The typically higher material strength of deformed bars substantially increases their ability to withstand the cracking which occurs during the development of tensile membrane action.
- The typically higher ductility of plain round bars increases their ability to withstand the cracking which occurs during the development of tensile membrane action.
- Surface geometry is a less significant factor than the mechanical properties of the steel in determining the ability of a reinforcing bar to withstand cracking during the development of tensile membrane action, under the conditions which have been investigated.
- Deformed bars perform comparably well to plain round bars in withstanding the cracking which occurs during the development of tensile membrane action, under the conditions which have been investigated.
- The common design assumption of perfect bond between reinforcement and concrete is appropriately, and not excessively, conservative in the design of hogging reinforcement of slabs utilising tensile membrane action in a fire scenario in the conditions investigated, provided the strain of the steel and concrete are accounted for. This appropriately conservative approach can be

recommended as typical design models are not sophisticated enough to accurately model the bond stress-slip interaction.

- The thermal gradient in a slab in a fire scenario helps the slab to withstand the cracking and rotation of the slab which occurs around the boundaries of the slab bay during the development of tensile membrane action without reinforcement rupture, under the conditions investigated. This is largely due to temperature changing the mechanical properties of concrete, rather than changes in bar or the bond response.
- With all else being equal, increased bar diameter is beneficial in the ability of a slab to withstand the rotation and cracking which occurs during the development of tensile membrane action without reinforcement rupture. This is due to the increased ratio of cross sectional area to bond area resulting in increased bond slip at the stage of rupture.
- As the point of crack rotation is closer to the level of the hogging reinforcement in a shallower slab, with all else being equal, shallower slabs are more able to withstand the rotation and cracking which occurs around the boundaries of a slab bay during the development of tensile membrane action than deeper slabs, within the range of depths investigated.

8.3 Further Work

The elevated temperature bond response was characterised for 8,10 and 12mm diameter bars, but larger bar diameters were not considered due to limitations of the test equipment. Further experimental pull-out testing at elevated temperatures should be conducted to characterise the bond response of modern deformed reinforcement with larger diameter bars. The results of such testing would allow the applicability of the modified local bond model for elevated temperatures presented in this research to be expanded for use with larger bar diameters. Additionally, the stress-slip response from these tests could be used in expanding the FE modelling which was conducted in this research to consider the behaviour of larger bar diameters. The performance of larger diameter bars in the crack-spanning situation considered in this research may be quite different as they may reach sufficient bond slip to pass the point of maximum bond stress in the crack-spanning situation considered in this research.

The bond strength retention of deformed bars at elevated temperatures depends on the retention of concrete compressive strength at elevated temperatures. As concrete compressive strength retention varies significantly with different aggregate types, bond strength retention at elevated temperatures will be dependent on aggregate type. It would be useful if bond strength retention at elevated temperatures could be investigated for a range of common concrete mixes, with different aggregate types so that these effects can be quantified for use in research models and, if necessary, the further development of the local bond model from this research.

Deformed bars will be, in theory, more prone to causing splitting of cover due to wedging action. Provided any splitting failure of the bond is at the top surface of a slab then this could allow more bond slip and less strain localisation in the reinforcement. Further research would be required to confirm this.

The scenarios which were modelled consider a situation where the slab of interest is located above a compartment fire. Tensile membrane action may be unable to form in a scenario where there is fire in the compartment below and in the compartment above as the hogging reinforcement would be subject to direct heating where cracks open up. Further research would, therefore, be needed to establish the suitability of utilising tensile membrane action in designs where a multi-floor fire is under consideration.

As this research has highlighted a concern about the ability of deeper composite slabs to develop tensile membrane action, it would be prudent for this to be investigated further. Additionally, there may be a minimum slab depth that is suitable for tensile membrane action as the hogging reinforcement may not have sufficient thermal insulation, resulting in weakening of the steel to the point of rupture around the slab boundaries. This was not directly investigated, but it should be considered in further experimental or numerical research.

9 References

Abd Elaty, M.A.A., 2014. Compressive strength prediction of Portland cement concrete with age using a new model. *HBRC journal*, 10(2), pp.145-155.

Anderberg, Y. and Thelandersson, S., 1976. Stress and deformation characteristics of concrete at high temperatures: experimental investigation and material behaviour model. Lund, Sweden: Lund institute of technology.

Aslani, F. and Bastami, M., 2011. Constitutive relationships for normal-and high-strength concrete at elevated temperatures. *ACI Materials Journal*, 108(4), p.355.

Bailey, C.G., Lennon, T. and Moore, D.B. 1999. The behaviour of full-scale steel-framed buildings subjected to compartment fires. *The Structural Engineer*, 77(8), pp.15-21.

Bailey, C.G., 2003. Efficient arrangement of reinforcement for membrane behaviour of composite floor slabs in fire conditions. *Journal of Constructional Steel Research*, 59(7), pp.931-949.

Bailey, C.G., 2004. Membrane action of slab/beam composite floor systems in fire. *Engineering Structures*, 26(12), pp.1691-1703.

Bailey, C.G. and Moore, D.B., 2000. The structural behaviour of steel frames with composite floorslabs subject to fire: Part 2: Design. *Structural Engineer*, 78(11), pp.28-33.

Bailey, C.G. and Toh, W.S., 2007. Small-scale concrete slab tests at ambient and elevated temperatures. *Engineering Structures*, 29(10), pp.2775-2791.

Bahr, O., Schaumann, P., Bollen, B. and Bracke, J., 2013. Young's modulus and Poisson's ratio of concrete at high temperatures: Experimental investigations. *Materials & Design*, 45, pp.421-429.

Bamonte, P.F. and Gambarova, P.G., 2007. High-bond bars in NSC and HPC: Study on size effect and on the local bond stress-slip law. *Journal of Structural Engineering*, 133(2), pp.225-234.

Bathe, KJ. 1996. Finite Element Procedures. Prentice Hall: Englewood Cliffs, NJ

Batson, A. 2013, How does fire affect concrete in shear?. MEng thesis. The University of Edinburgh.

Bergheau, J.M. and Fortunier, R., 2013. *Finite element simulation of heat transfer*. John Wiley & Sons.

Botte, W., Caspeelee, R., Gouverneur, D. and Taerwe, L., 2014, July. Influence of membrane action on robustness indicators and a global resistance factor design. In *Proceedings of the 7th Int. Conf. Bridge Maintenance, Safety, Management and Life Extension*, Shanghai, China (pp. 7-11).

British Steel. 1999. The Behaviour of Multi-storey Steel Framed Buildings in Fire. British Steel. Moorgate, Rotherham, UK.

British Standards Institution (BSI). 1978. BS 4449: 1978. Specification for hot rolled steel bars for the reinforcement of concrete. UK:BSI

British Standards Institution (BSI). 2002. Eurocode 1. Actions on structures. General actions. Actions on structures exposed to fire. UK:BSI

British Standards Institution (BSI). 2004. BS EN 10025-1:2004 Hot rolled products of structural steels - General technical delivery conditions. UK:BSI

British Standards Institution (BSI). 2005a. Eurocode 3. Design of steel structures - General rules - Structural fire design. BSI:UK

British Standards Institution (BSI). 2005b. Eurocode 4. Design of composite steel and concrete structures - General rules. Structural fire design. UK:BSI

British Standards Institution (BSI). 2005c. BS 4449:2005, Steel for the reinforcement of concrete — Weldable reinforcing steel — Bar, coil and decoiled product — Specification. UK:BSI

British Standards Institution (BSI). 2002. BS EN 12390-6:2009 Testing hardened concrete - Tensile splitting strength of test specimens. UK:BSI

British Standards Institution (BSI), 2014. BS 476-20:1987. Fire tests on building materials and structures - Method for determination of the fire resistance of elements of construction (general principles). UK:BSI

British Standards Institution (BSI). 2015. BS 9991:2015 Fire safety in the design, management and use of residential buildings. Code of practice. UK:BSI

British Standards Institution (BSI). 2019. Eurocode 2. Design of concrete structures - General rules. Structural fire design. UK:BSI

British Standards Institution (BSI). 2020. BS EN 1363-1:2020 Fire resistance tests. General requirements. UK:BSI

Buchanan, A.H. and Abu, A.K., 2017. Structural design for fire safety. John Wiley & Sons.

Cairns, J. and Plizzari, G.A., 2003. Towards a harmonised European bond test. *Materials and Structures*, 36(8), pp.498-506.

Cairns, J., 2020. Local bond–slip model for plain surface reinforcement. *Structural Concrete*, 22(2), pp. 666-675.

Cameron, N. and Usmani, A., 2005. A New Design Method to Determine the Membrane Capacity of Laterally Restrained Composite Floor slabs in Fire, Part 1: Theory and Method; Part 2: Validation. *The Structural Engineer*, 83, pp.28-33.

Castillo, C. and Durrani, A.J., 1990. Effect of transient high-temperature on high-strength concrete. *ACI Materials Journal*, 87(6), pp.653-653.

Cruz, C.R. and Gillen, M., 1980. Thermal expansion of Portland cement paste, mortar and concrete at high temperatures. *Fire and materials*, 4(2), pp.66-70.

Dassault Systèmes. 2012. Abaqus Analysis User's Manual. Providence, RI, USA.

Diederichs, U. and Schneider, U., 1981. Bond strength at high temperatures. *Magazine of Concrete Research*, 33(115), pp.75-84.

Diederichs, U. and Schneider, U., 1982. Changes in Bond Behaviour due to Elevated Temperatures. *International Conference on Bond in Concrete*, Paisley College of Technology, Scotland. 14 to 16 June.

- Dong, Y.L. and Zhu, C.J., 2011. Limit load carrying capacity of two-way slabs with two edges clamped and two edges simply supported in fire. *Journal of Structural Engineering*, 137(10), pp.1182-1192.
- Fayyad, T.M. and Lees, J.M. Experimental investigation of crack propagation and crack branching in lightly reinforced concrete beams using digital image correlation. *Engineering Fracture Mechanics*. 182 (2017) pp. 487-505.
- Flint, G., Lamont, S., Lane, B., Sarrazin, H., Lim, L., Rini, D. and Roben, C., 2013. Recent lessons learned in structural fire engineering for composite steel structures. *Fire technology*, 49(3), pp.767-792.
- Foster, S.J., Bailey, C.G., Burgess, I.W. and Plank, R.J., 2004. Experimental behaviour of concrete floor slabs at large displacements. *Engineering structures*, 26(9), pp.1231-1247.
- Franssen, J.M., 2005. SAFIR: A thermal/structural program for modeling structures under fire. *Engineering Journal*, 42(3).
- Gernay, T., 2012. Effect of transient creep strain model on the behavior of concrete columns subjected to heating and cooling. *Fire technology*, 48(2), pp.313-329.
- Gernay, T. and Franssen, J.M., 2012. A formulation of the Eurocode 2 concrete model at elevated temperature that includes an explicit term for transient creep. *Fire Safety Journal*, 51, pp.1-9.
- Gillie, M., Usmani, A.S. and Rotter, J.M., 2001. A structural analysis of the first Cardington test. *Journal of Constructional Steel Research*, 57(6), pp.581-601.
- Giroldo, F. and Bailey, C.G., 2008. Experimental bond behaviour of welded mesh reinforcement at elevated temperatures. *Magazine of Concrete Research*, 60(1), pp.23-31.
- Hager, I., Tracz, T., Śliwiński, J. and Krzemień, K., 2015. The influence of aggregate type on the physical and mechanical properties of high-performance concrete subjected to high temperature. *Fire and materials*, 40(5), pp.668-682.

Han, J., Wang, S., Chen, Y. and Cao, C., 2019. Analytical derivation of rib bearing angle of reinforcing bar subject to axial loading. *Magazine of concrete research*, 71(4), pp.175-183.

Hertz, K., 1982. The anchorage capacity of reinforcing bars at normal and high temperatures. *Magazine of concrete research*, 34(121), pp.213-220.

International Federation for Structural Concrete. 1993. CEB-FIP Model Code 1990. fib.

International Federation for Structural Concrete, 2000. Bond of reinforcement in concrete. *FIB Bulletin No. 10*. fib.

International Federation for Structural Concrete. 2013. fib Model Code for Concrete Structures 2010. fib.

International Organization for Standardization (ISO). 2014. ISO 834-10:2014. Fire resistance tests — Elements of building construction — Part 10: Specific requirements to determine the contribution of applied fire protection materials to structural steel elements.

Izzuddin, B.A., Tao, X.Y. and Elghazouli, A.Y., 2004. Realistic modeling of composite and reinforced concrete floor slabs under extreme loading. I: Analytical method. *Journal of Structural Engineering*, 130(12), pp.1972-1984.

Jiang, J. and Li, G.Q., 2018. Parameters affecting tensile membrane action of reinforced concrete floors subjected to elevated temperatures. *Fire safety journal*, 96, pp.59-73.

Jiang, J., Qi, H., Lu, Y., Li, G.Q., Chen, W. and Ye, J., 2022. A state-of-the-art review on tensile membrane action in reinforced concrete floors exposed to fire. *Journal of Building Engineering*, 45, 103502

Jiang, J. and Usmani, A., 2013. Modeling of steel frame structures in fire using OpenSees. *Computers & Structures*, 118, pp.90-99.

Jirsa, J.O., Breen, J.E., Luke, J.J. and Hamad, B.S., 1982. Effect of casting position on bond. *International Conference on Bond in Concrete* (pp. 300-307). Paisley College of Technology Paisley Scotland.

Johnson, R.P., 2018. *Composite Structures of Steel and Concrete: beams, slabs, columns and frames for buildings*. John Wiley & Sons.

Ka, S.B., Han, S.J., Lee, D.H., Choi, S.H., Oh, Y.H. and Kim, K.S., 2018. Bond strength of reinforcing bars considering failure mechanism. *Engineering Failure Analysis*, 94, pp.327-338.

Kankam, C.K., 2003. A routine method for measuring bond stress, steel strain and slip in reinforced concrete beams at service loads. *Magazine of concrete research*, 55(1), pp.85-93.

Khoury, G.A., 2000. Effect of fire on concrete and concrete structures. *Progress in Structural Engineering and Materials*, 2(4), pp.429-447.

Kodur, V.K.R., 2005. Guidelines for fire resistance design of high-strength concrete columns. *Journal of fire protection engineering*, 15(2), pp.93-106

Kodur, V., Dwaikat, M. and Fike, R., 2010. High-temperature properties of steel for fire resistance modeling of structures. *Journal of Materials in Civil Engineering*, 22(5), pp.423-434.

Kodur, V.K.R., Naser, M., Pakala, P. and Varma, A., 2013. Modeling the response of composite beam–slab assemblies exposed to fire. *Journal of Constructional Steel Research*, 80, pp.163-173.

Kucharska, M. and Jaskowska-Lemanska, J., 2019. Properties of a bond between the steel reinforcement and the new generation concretes—a review. *IOP Conference Series: Materials Science and Engineering* (Vol. 603, No. 4, p. 042057). IOP Publishing.

Lange, D. and Jansson, R., 2014. A comparison of an explicit and an implicit transient strain formulation for concrete in fire. *Fire Safety Science*, 11, pp.572-583.

Law, A., 2010. The assessment and response of concrete structures subject to fire. Doctoral dissertation, The University of Edinburgh.

Law, A., Gillie, M. and Pankaj, P., 2009. Incorporation of Load Induced Thermal Strain in Finite Element Models. *Acta Polytechnica*, 49(1).

- Lee, J. and Fenves, G.L., 1998. Plastic-damage model for cyclic loading of concrete structures. *Journal of engineering mechanics*, 124(8), pp.892-900.
- Li, G.Q., Zhang, N. and Jiang, J., 2017. Experimental investigation on thermal and mechanical behaviour of composite floors exposed to standard fire. *Fire Safety Journal*, 89, pp.63-76.
- Lim, L., Buchanan, A., Moss, P. and Franssen, J. 2004. Numerical modelling of two-way reinforced concrete slabs in fire. *Engineering Structures*, 26, pp 1081-1091.
- Lubliner, J., Oliver, J., Oller, S. and Onate, E., 1989. A plastic-damage model for concrete. *International Journal of solids and structures*, 25(3), pp.299-326.
- Lutz, L.A. and Gergely, P., 1967. Mechanics of bond and slip of deformed bars in concrete. *Journal Proceedings* Vol. 64, No. 11, pp. 711-721.
- Malik, M., Bhattacharyya, S.K. and Barai, S.V., 2021. Thermal and mechanical properties of concrete and its constituents at elevated temperatures: A review. *Construction and Building Materials*, 270, p.121398.
- Mathey, R.G. and Watstein, D., 1961. Investigation of bond in beam and pull-out specimens with high-yield-strength deformed bars. *Journal Proceedings* (Vol. 57, No. 3, pp. 1071-1090).
- Meacham.B., Bowen.R., Traw.J. and Moore.A., 2005. Performance-based building regulation: current situation and future needs, *Building research and information*. 33(2), p. 91–106
- Menzel, C.A., 1939. Some factors influencing results of pull-out bond tests. In *Journal Proceedings* (Vol. 35, No. 6, pp. 517-542).
- Mo, Y.L. and Chan, J., 1996. Bond and slip of plain rebars in concrete. *Journal of materials in Civil Engineering*, 8(4), pp.208-211.
- Monte, F.L., Kalaba, N. and Bamonte, P., 2017. On the extension of a plastic-damage model to high temperature and fire. In *IFireSS 2017-2nd International Fire Safety Symposium* (pp. 703-710).

- Morley, P.D., 1982. Effects of elevated temperature on bond in reinforced concrete. Doctoral dissertation, The University of Edinburgh.
- Morley, P.D. and Royles, R., 1983. Response of the bond in reinforced concrete to high temperatures. *Magazine of Concrete Research*, 35(123), pp.67-74.
- Nielsen, C.V., Pearce, C.J. and Bićanić, N., 2002. Theoretical model of high temperature effects on uniaxial concrete member under elastic restraint. *Magazine of Concrete Research*, 54(4), pp.239-249.
- Newman, G.M., Robinson, J.T. and Bailey, C.G., Fire Safe Design: A new Approach to Multi-Story Steel-Framed Buildings.: The Steel Construction Institute; 2006. SCI Publication P, 288.
- Nguyen, T.T., Tan, K.H. and Burgess, I.W., 2015. Behaviour of composite slab-beam systems at elevated temperatures: Experimental and numerical investigation. *Engineering Structures*, 82, pp.199-213.
- Noumowe, A.N., 1996. Thermal stresses and water vapour pressure of high performance concrete at high temperature. *In 4th Int. Symp. on Utilization of High-strength/High-performance Concrete*, Paris (Vol. 2, pp. 561-570).
- O'Connor, M., Kirby, B.R. and Martin, D.M. 2003. Behaviour of a multi-storey composite steel framed building in fire. *Structural Engineer*, 81(2), pp.27-36.
- Omer, E., Izzuddin, B.A. and Elghazouli, A.Y., 2009. Failure of lightly reinforced concrete floor slabs with planar edge restraints under fire. *Journal of structural engineering*, 135(9), pp.1068-1080.
- Park, R., 1964. Tensile membrane behaviour of uniformly loaded rectangular reinforced concrete slabs with fully restrained edges. *Magazine of Concrete Research*, 16(46), pp. 39-44.
- Petersson, P.E., 1981. Crack growth and development of fracture zones in plain concrete and similar materials. Doctoral dissertation, Lund University.

Rehm, G., Ueber die Grundlagen des Verbundes zwischen Stahl und Beton (The fundamentals of bond between steel reinforcement and concrete. *Deutscher Ausschuß für Stahlbeton* (DAf St), 138, 1961.

Reynolds, G.C., 1982. Bond strength of deformed bars in tension. *Cement and Concrete Association Technical Report*, 548. Slough, England.

RILEM. 1994. Technical Recommendations for the testing and use of construction materials. CRC Press

Royles, R. and Morley, P.D., 1983. Further responses of the bond in reinforced concrete to high temperatures. *Magazine of Concrete Research*, 35(124), pp.157-163.

Snowdon, L.G., 1970. Classifying reinforcing bars for bond strength. Building Research Station. *Garston*. Nov. 1970. pp.40. CP36/70.

Society of Fire Protection Engineers. 2004. Engineering guide - fire exposures to structural elements. SFPE

Spinardi, G., 2016. Fire safety regulation: Prescription, performance, and professionalism. *Fire Safety Journal*, 80, pp.83-88.

Tepfers, R., 1979. Cracking of concrete cover along anchored deformed reinforcing bars. *Magazine of concrete research*, 31(106), pp.3-12.

Terro, M.J., 1998. Numerical modeling of the behavior of concrete structures in fire. *ACI Structural Journal*, 95, pp.183-193.

Teychenné, D.C., Franklin, R.E., Erntroy, H.C. and Marsh, B.K., 1997. BRE *Design of normal concrete mixes*. HM Stationery Office.

Torelli, G., Mandal, P., Gillie, M. and Tran, V.X., 2016. Concrete strains under transient thermal conditions: A state-of-the-art review. *Engineering Structures*, 127, pp.172-188.

Torero, J.L. and Lane, B., 2004. The Changing Face of Structural Design for Fire. *International Fire Buyers Guide*, August, 2004

Tufail, M., Shahzada, K., Gencturk, B. and Wei, J., 2017. Effect of elevated temperature on mechanical properties of limestone, quartzite and granite concrete. *International Journal of Concrete Structures and Materials*, 11(1), pp.17-28.

Wahid, N. Stratford, T. Bisby, L., 2019. Calibration of Concrete Damage Plasticity Model Parameters for High Temperature Modelling of Reinforced Concrete Flat Slabs in Fire. *Applications of Structural Fire Engineering*, 13-14 June 2019, Singapore

Walker, PR. Batayneh, M. K. and Regan, P. E.. 1997. Bond strength tests on deformed reinforcement in normal weight concrete. *Materials and Structures / Matériaux et Constructions*, 30, August-September, 424-429

Weigler, H. and Fischer, R., 1972. Influence of high temperatures on strength and deformation of concrete. *Special Publication*, 34, pp.481-494.

Wood, R.H., 1961. Plastic and elastic design of slabs and plates: with particular reference to reinforced concrete floor slabs. Ronald Press Company.

Xu, Y., Wong, Y.L., Poon, C.S. and Anson, M., 2003. Influence of PFA on cracking of concrete and cement paste after exposure to high temperatures. *Cement and Concrete Research*, 33(12), pp.2009-2016.

Zheng, Y., Robinson, D., Taylor, S., Cleland, D. and Shaat, A., 2008. Analysis of compressive membrane action in concrete slabs. *Proceedings of the Institution of Civil Engineers-Bridge Engineering*, 161(1) pp. 21-31.

Zienkiewicz, O.C. and Taylor, R.L., 2005. *The finite element method for solid and structural mechanics*. Elsevier.

Appendix A - stress-slip curves with best-fit parameter values in fib model

The figures included here present the stress-slip curves which are calculated from the fib model using the parametric values which were determined to be of best-fit to the experimental data by regression analysis. The experimental data used was the average of results for that temperature and bar diameter.

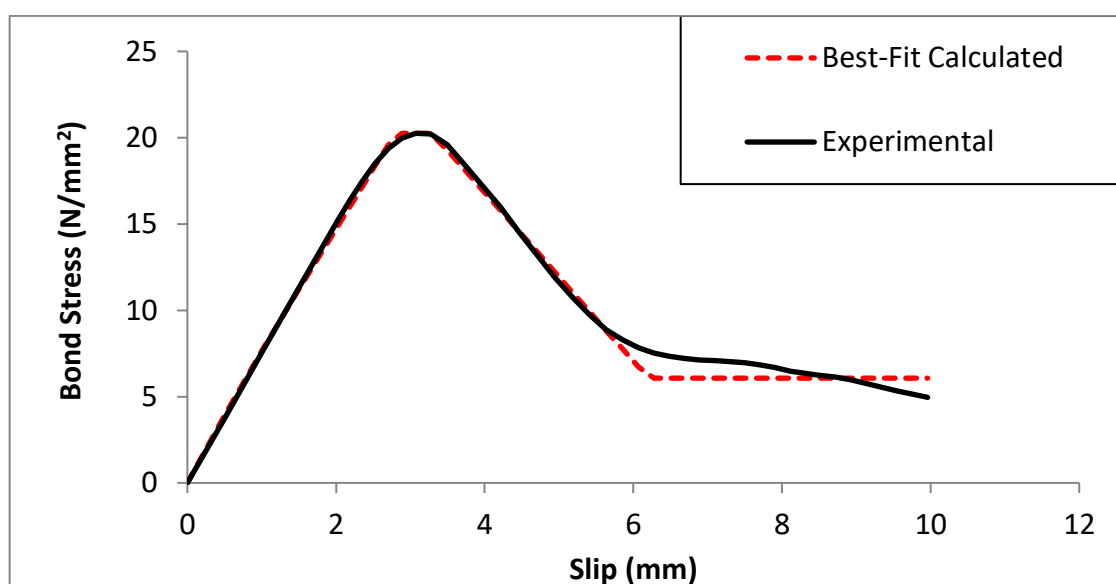


Figure A.1. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 8mm diameter bar at 15°C

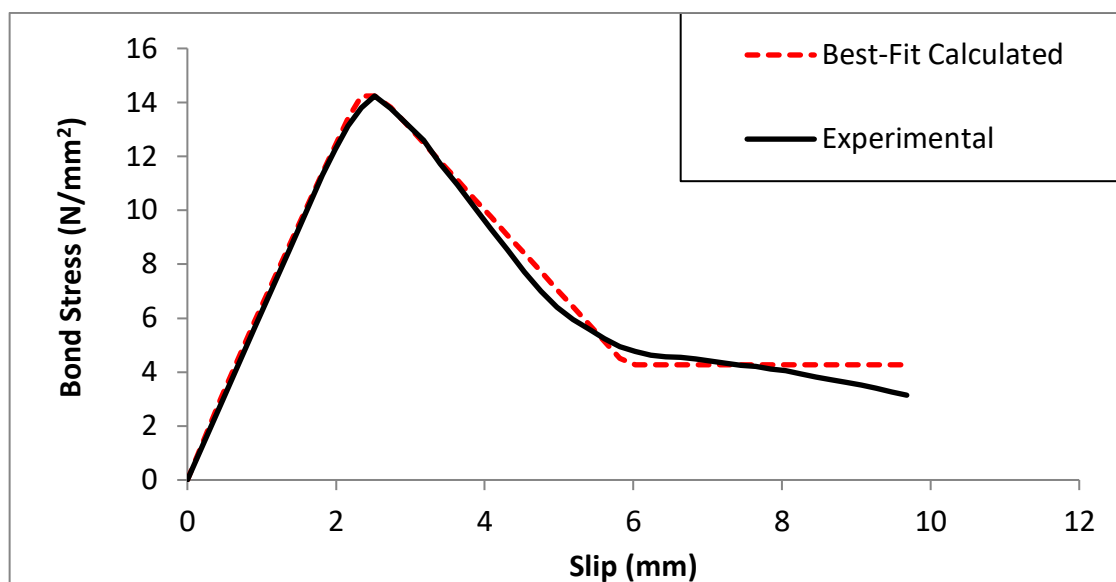


Figure A.2. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 8mm diameter bar at 110°C

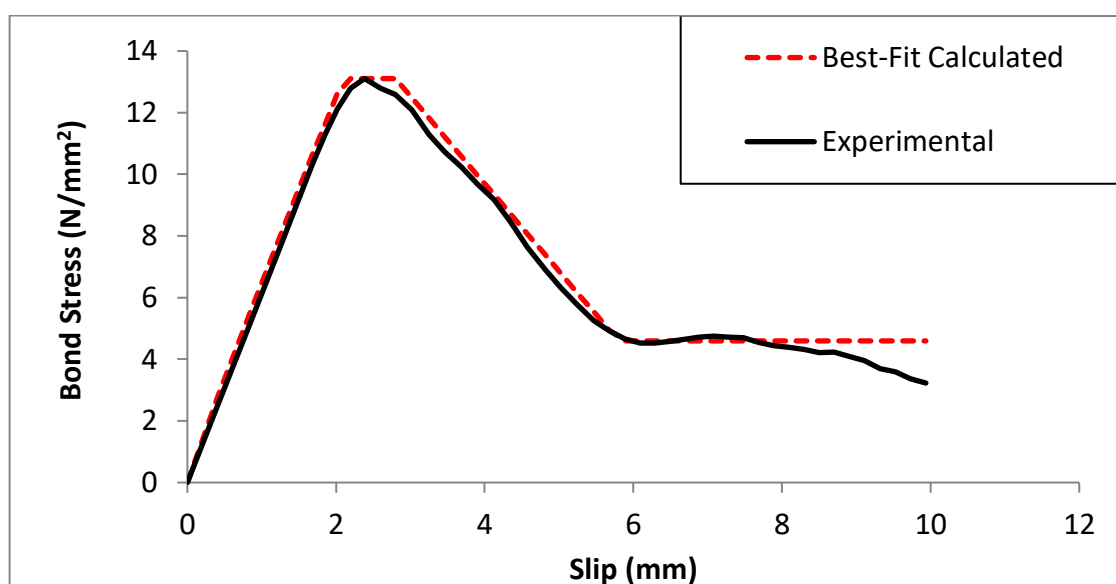


Figure A.3. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 8mm diameter bar at 210°C

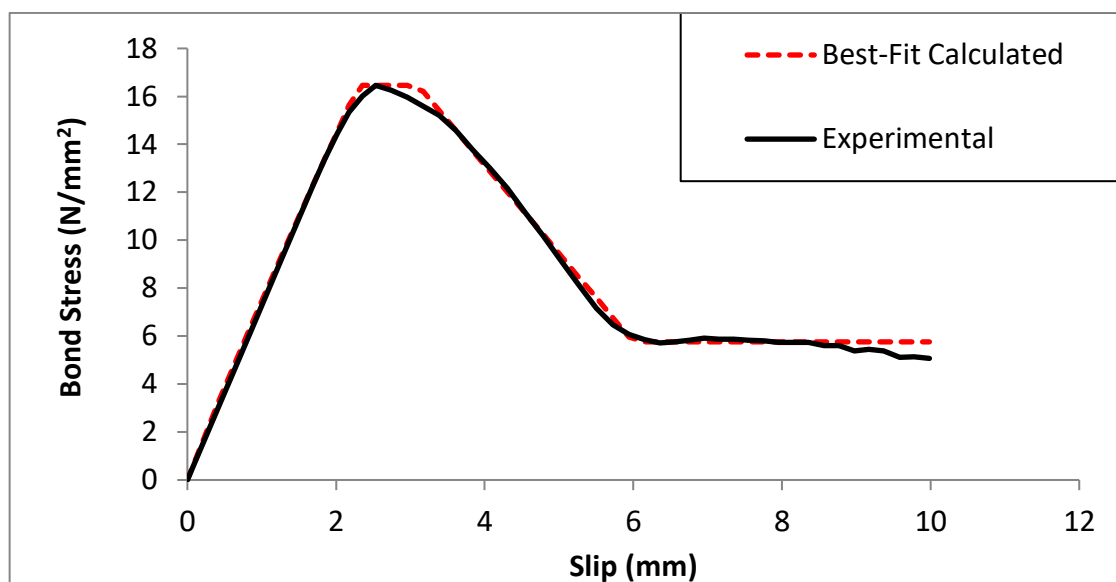


Figure A.4. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 8mm diameter bar at 290°C

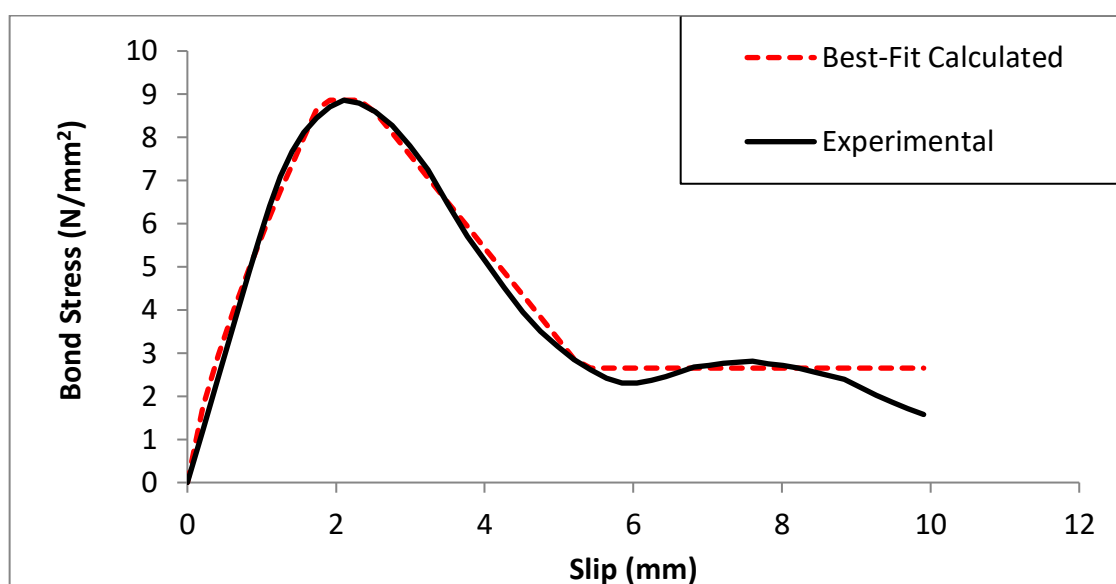


Figure A.5. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 8mm diameter bar at 570°C

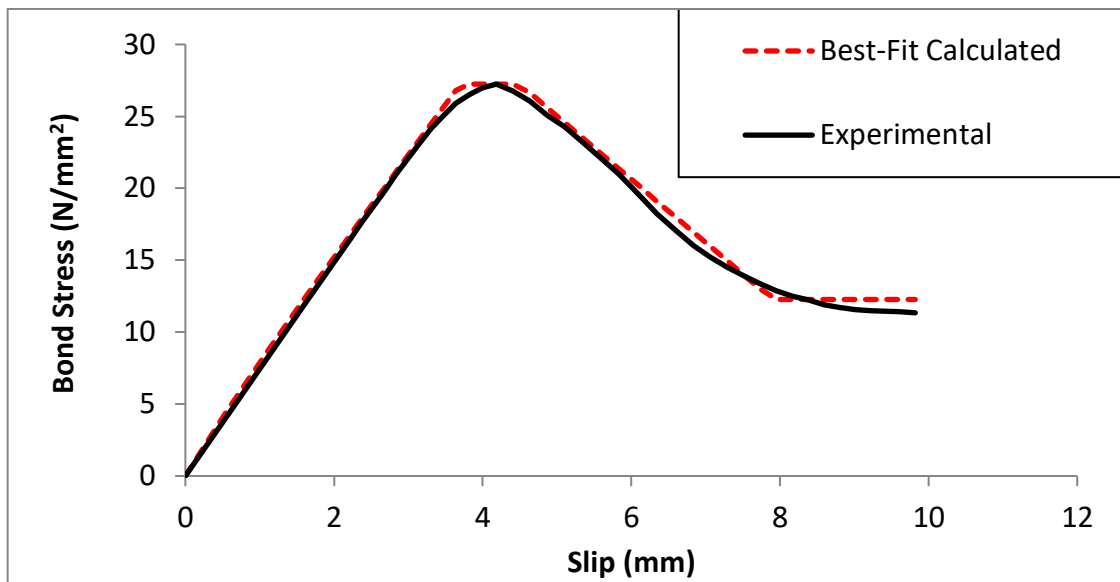


Figure A.6. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 10mm diameter bar at 15°C

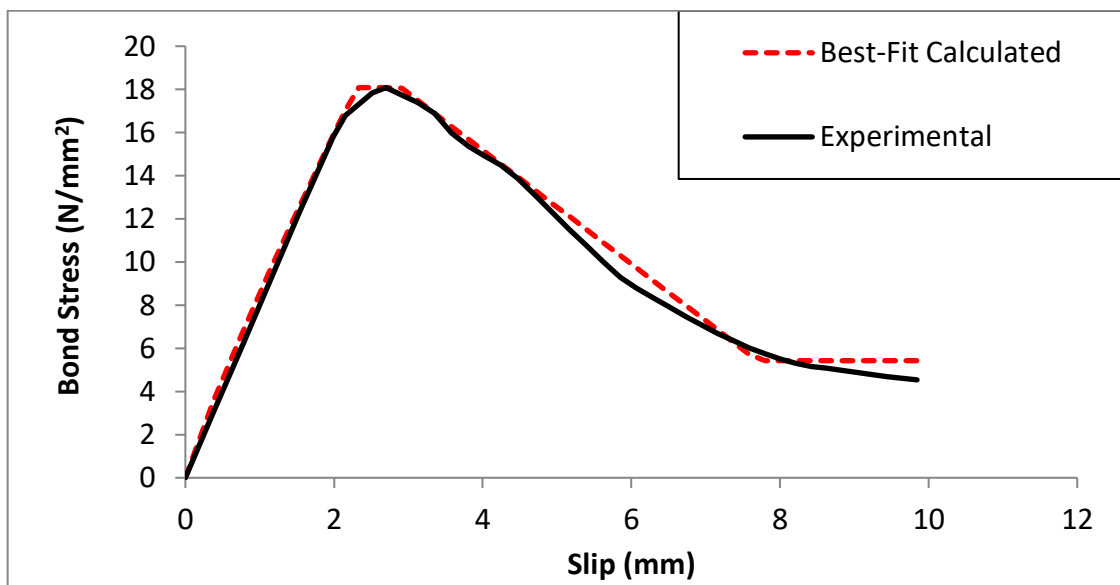


Figure A.7. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 10mm diameter bar at 110°C

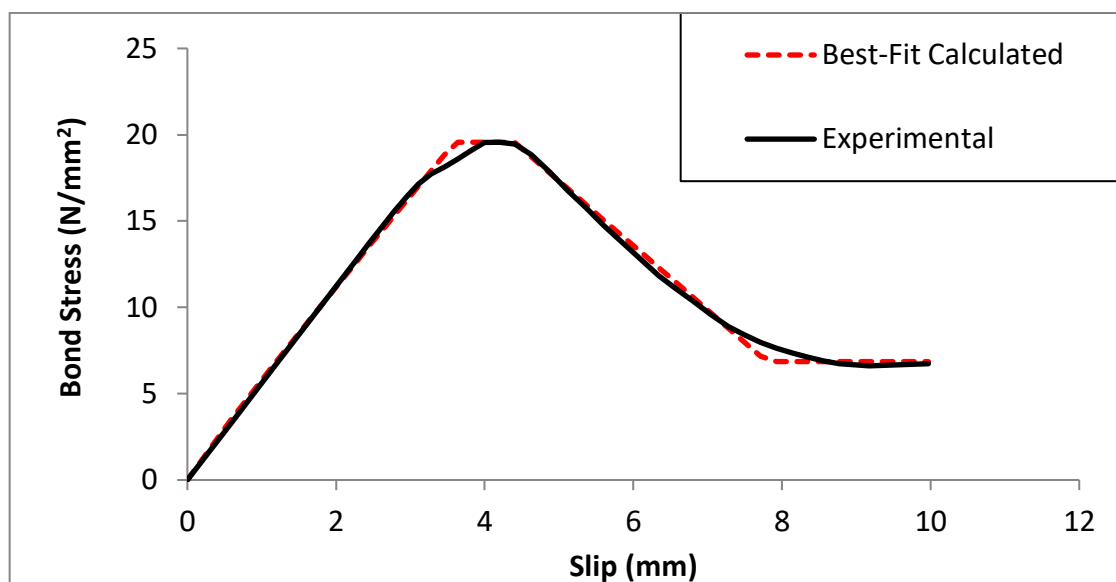


Figure A.8. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 10mm diameter bar at 210°C

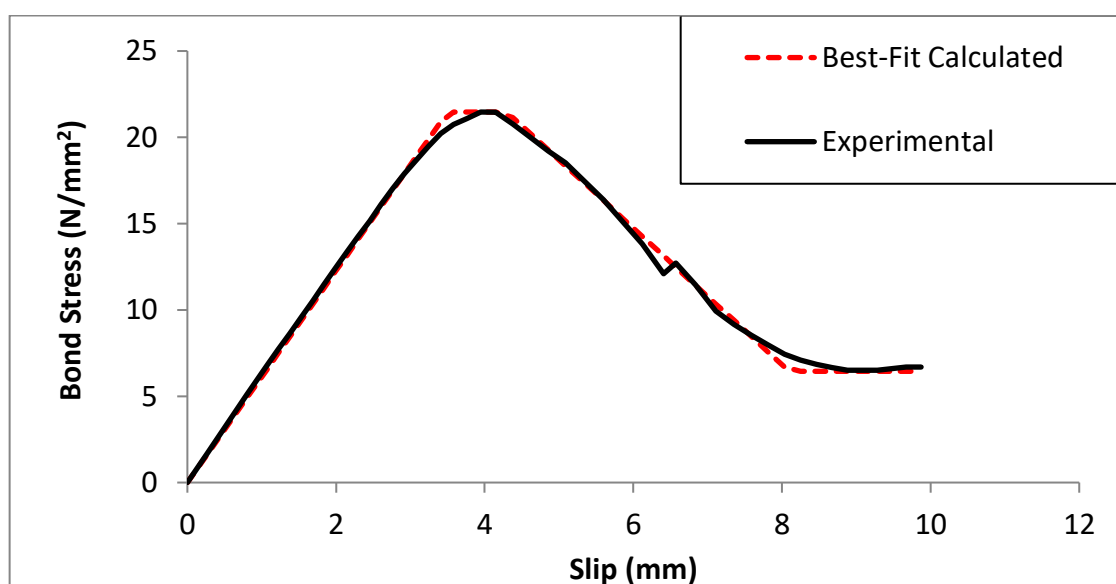


Figure A.9. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 10mm diameter bar at 290°C

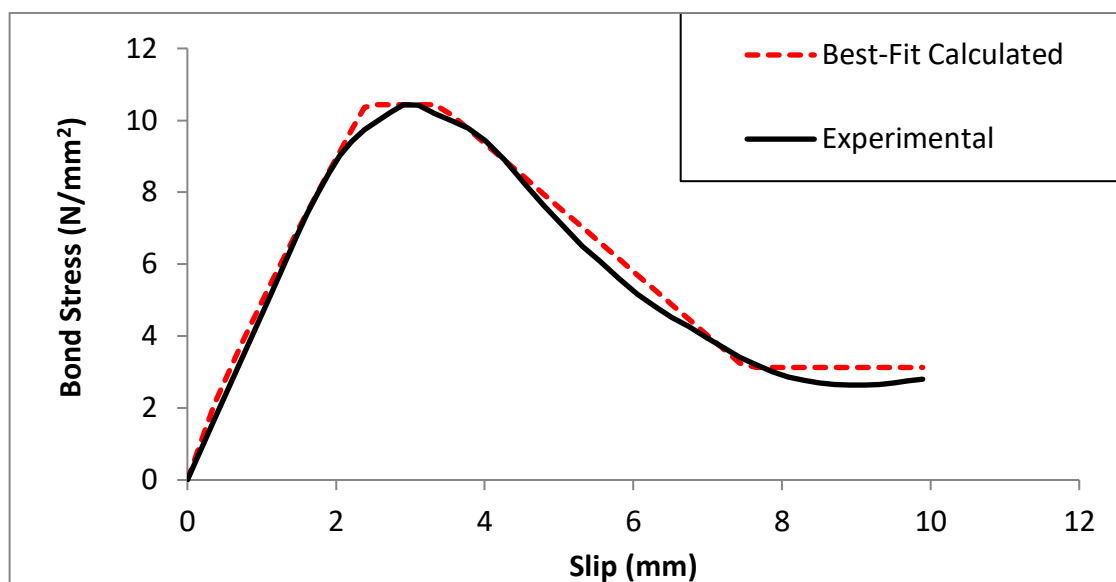


Figure A.10. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 10mm diameter bar at 570°C

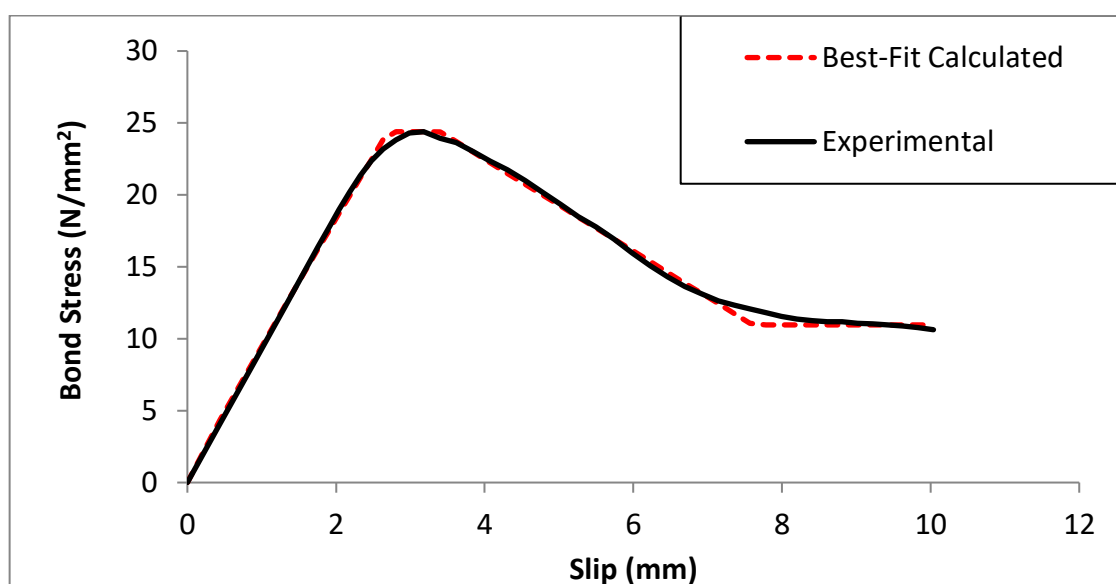


Figure A.11. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 12mm diameter bar at 15°C

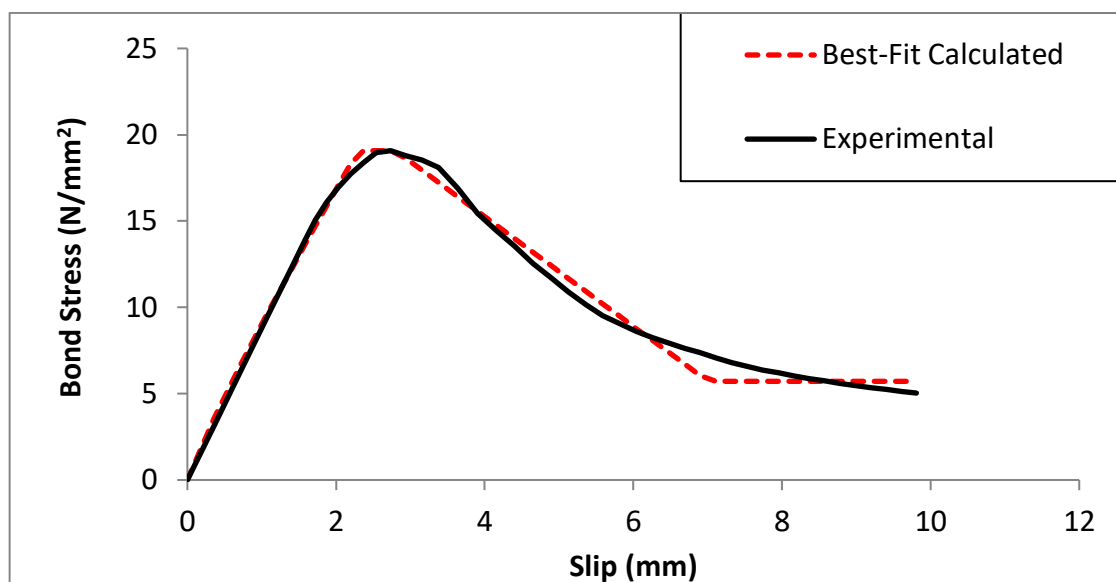


Figure A.12. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 12mm diameter bar at 110°C

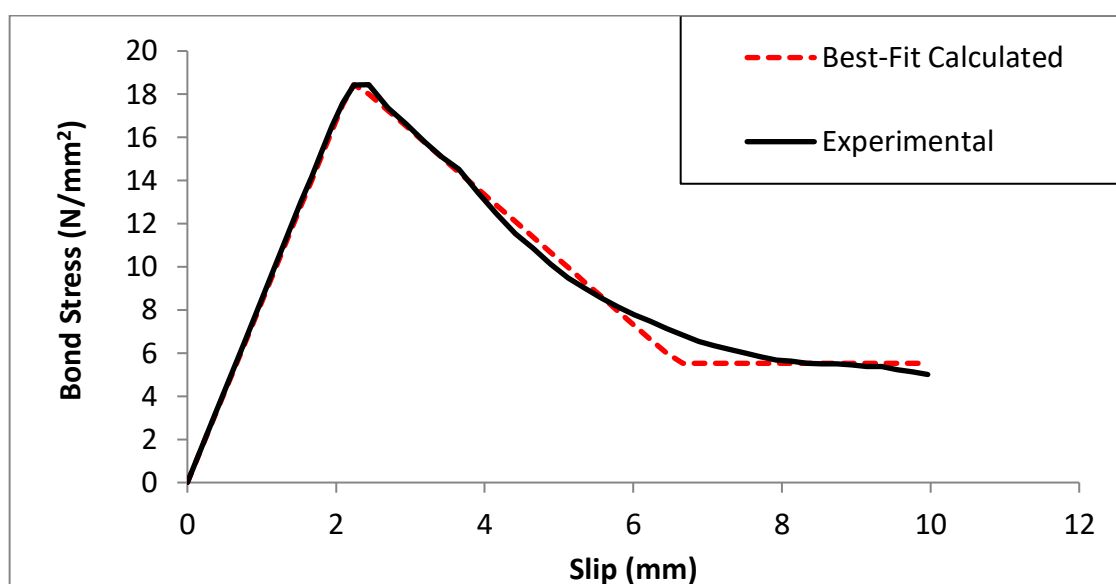


Figure A.13. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 12mm diameter bar at 210°C

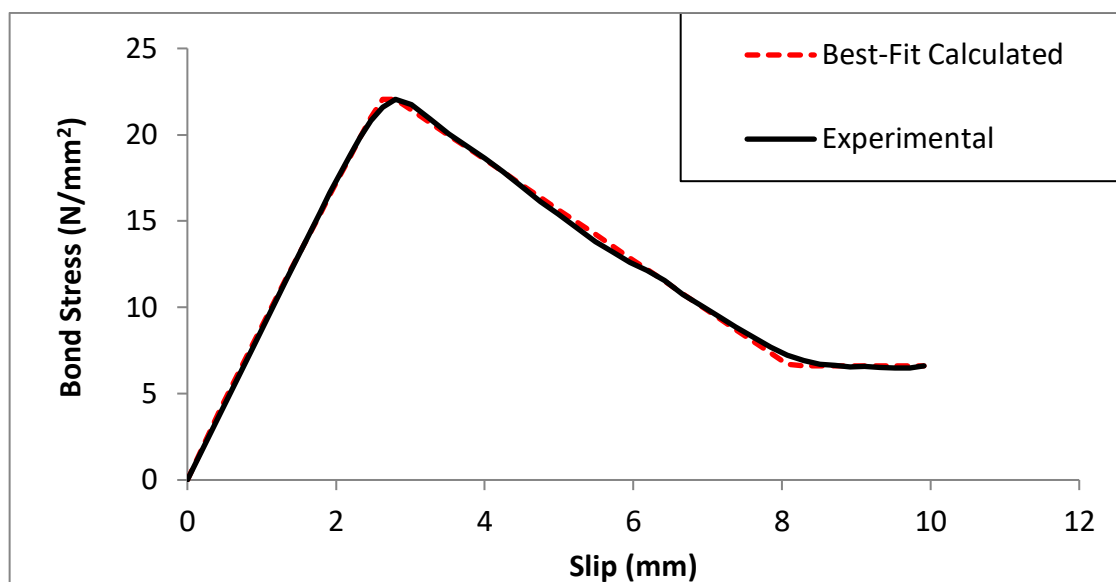


Figure A.14. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 12mm diameter bar at 290°C

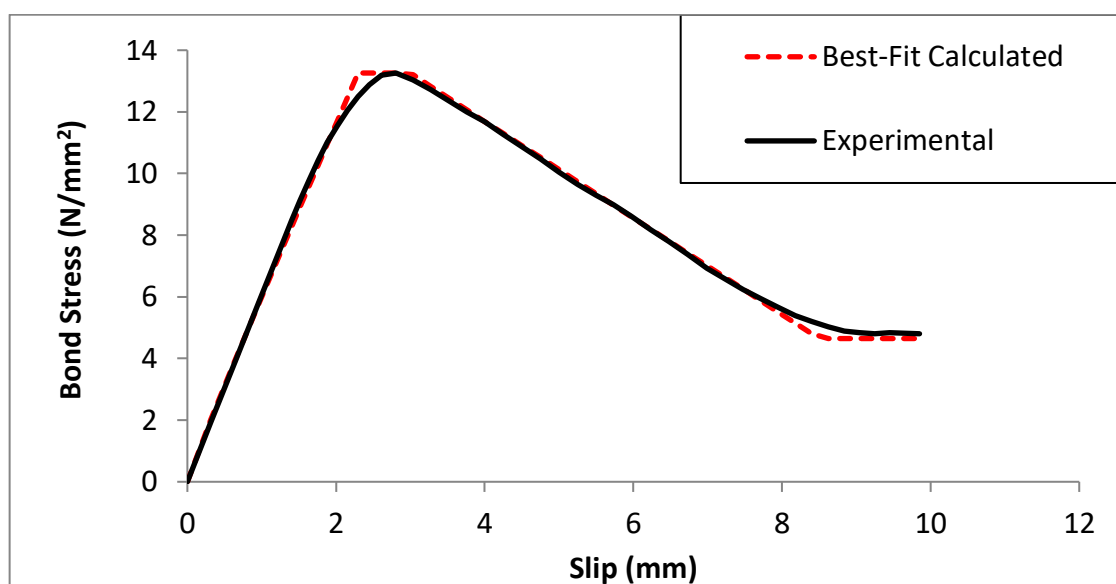


Figure A.15. Comparison of experimental and calculated stress-slip curve using best-fit values of parameters for 12mm diameter bar at 570°C

Appendix B - Tables of best-fit parameter values for fib model

The tables included here present the values of the parameters in the fib model which were determined to be of best-fit to the experimental data by regression analysis.

Bar Diameter (mm)	8	8	8	8	8
Temperature (°C)	15	107	207	291	573
α	0.95	0.95	0.95	0.95	0.75
α Normalised*	1.00	1.00	1.00	1.00	0.79
s_1 (mm)	2.8	2.3	2.1	2.3	1.8
s_1 Normalised*	1.00	0.82	0.75	0.82	0.64
s_2 (mm)	3.3	2.6	2.8	3.1	2.4
s_2 Normalised*	1.00	0.79	0.85	0.94	0.73
s_3 (mm)	6.2	5.9	5.8	6	5.3
s_3 Normalised*	1.00	0.95	0.94	0.97	0.85
T_{bmax} (N/mm ²)	20.3	14.2	13.1	16.5	8.9
T_{bmax} Normalised*	1.00	0.70	0.65	0.81	0.44
T_f (N/mm ²)	6.1	4.3	4.6	5.8	2.7
T_f Normalised*	1.00	0.70	0.75	0.95	0.44
T_f / T_{bmax}	0.3	0.3	0.35	0.35	0.3

Table B.1. Best-fit values of each parameter in the fib bond model for 8mm diameter deformed bar, as derived from regression analysis

Bar Diameter (mm)	10	10	10	10	10
Temperature (°C)	15	107	207	291	573
α	0.95	0.9	0.95	1	0.85
α Normalised*	1.00	0.95	1.00	1.05	0.89
s_1 (mm)	3.7	2.3	3.6	3.5	2.4
s_1 Normalised*	1.00	0.62	0.97	0.95	0.65
s_2 (mm)	4.5	2.9	4.4	4.3	3.4
s_2 Normalised*	1.00	0.64	0.98	0.96	0.76
s_3 (mm)	7.9	7.7	7.8	8.1	7.5
s_3 Normalised*	1.00	0.97	0.99	1.03	0.95
T_{bmax} (N/mm ²)	27.2	18.1	19.6	21.5	10.4
T_{bmax} Normalised*	1.00	0.66	0.72	0.79	0.38
T_f (N/mm ²)	12.3	5.4	6.8	6.4	3.1
T_f Normalised*	1.00	0.44	0.56	0.53	0.26
T_f / T_{bmax}	0.45	0.3	0.35	0.3	0.3

Table B.2. Best-fit values of each parameter in the fib bond model for 10mm diameter deformed bar, as derived from regression analysis

Bar Diameter (mm)	12	12	12	12	12
Temperature (°C)	15	107	207	291	573
α	0.95	0.9	1	0.95	0.95
α Normalised*	1.00	0.95	1.05	1.00	1.00
s_1 (mm)	2.7	2.3	2.2	2.6	2.3
s_1 Normalised*	1.00	0.85	0.81	0.96	0.85
s_2 (mm)	3.4	2.8	2.3	2.8	3.0
s_2 Normalised*	1.00	0.82	0.68	0.82	0.88
s_3 (mm)	7.6	7	6.6	8.1	8.5
s_3 Normalised*	1.00	0.92	0.87	1.07	1.12
T_{bmax} (N/mm ²)	24.4	19.1	18.4	22.1	13.3
T_{bmax} Normalised*	1.00	0.78	0.76	0.90	0.54
T_f (N/mm ²)	11.0	5.7	5.5	6.6	4.6
T_f Normalised*	1.00	0.52	0.50	0.60	0.42
T_f / T_{bmax}	0.45	0.3	0.3	0.3	0.35

Table B.3. Best-fit values of each parameter in the fib bond model for 12mm diameter deformed bar, as derived from regression analysis

*Normalised relative to value at 15°C

Appendix C - Relationship between fib model parameters and temperature

The figures in this appendix show fib model parameter values which were determined to have the best-fit with the experimental data from regression analysis. These best-fit values are plotted against temperature to show the relationship of each parameter with temperature.

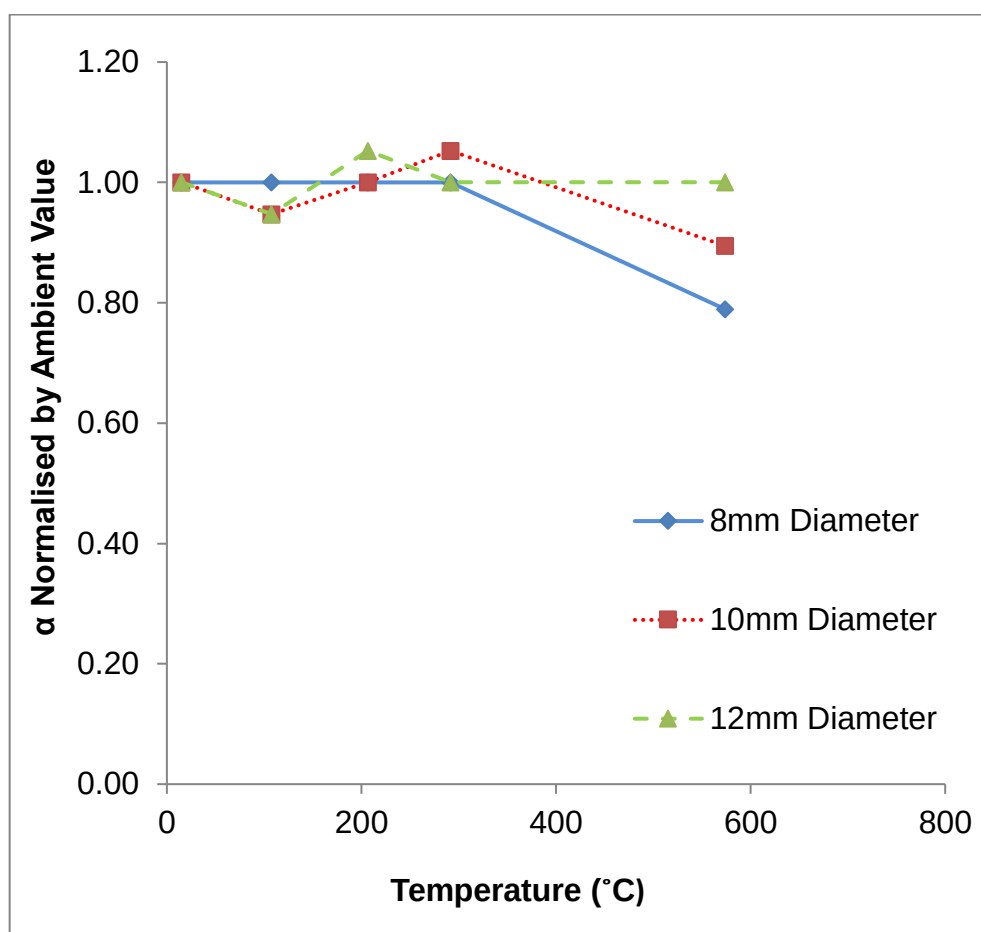


Figure C.1. Best-fit values of α in fib model from regression analysis at different temperatures, normalised relative to value at 15°C

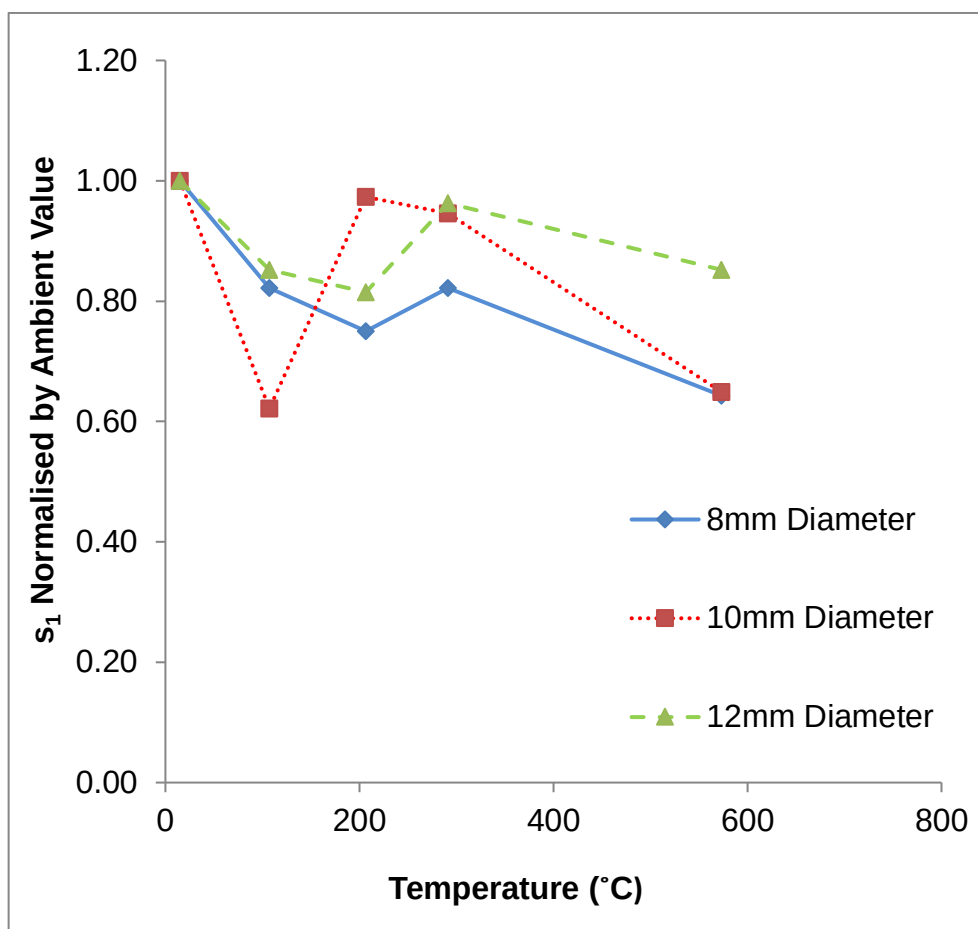


Figure C.2. Best-fit values of s_1 in fib model from regression analysis at different temperatures, normalised relative to value at 15°C

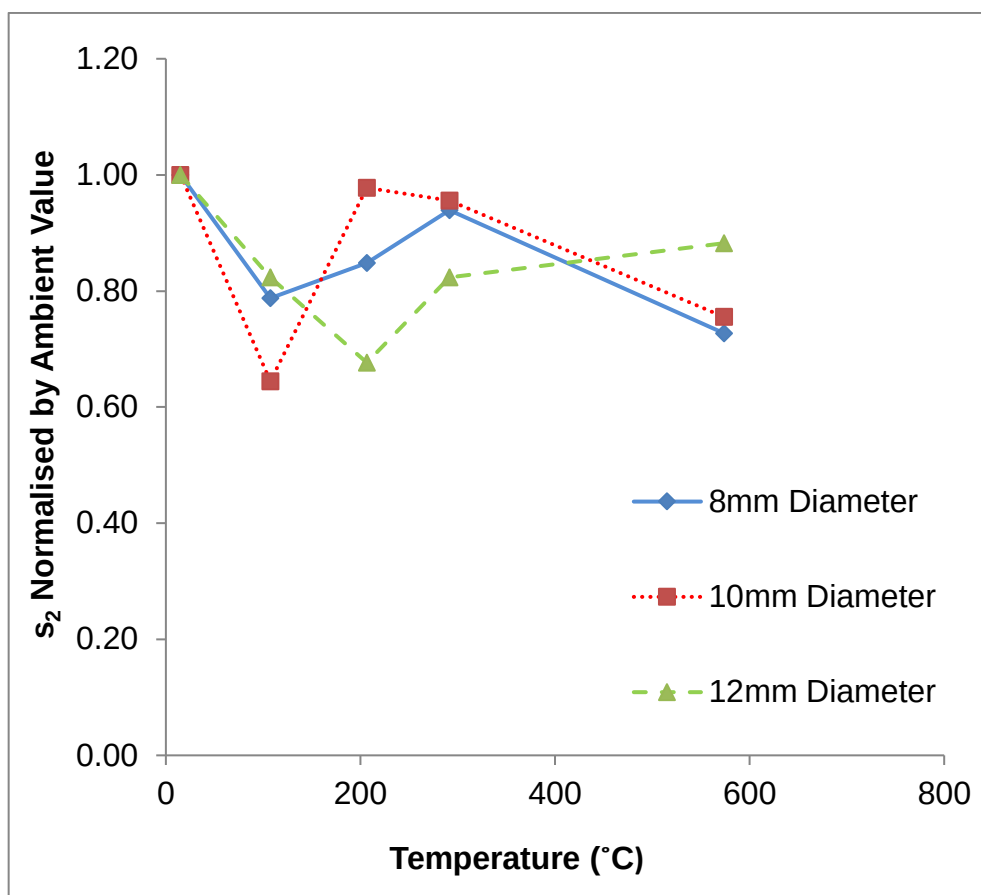


Figure C.3. Best-fit values of s_2 in fib model from regression analysis at different temperatures, normalised relative to value at 15°C

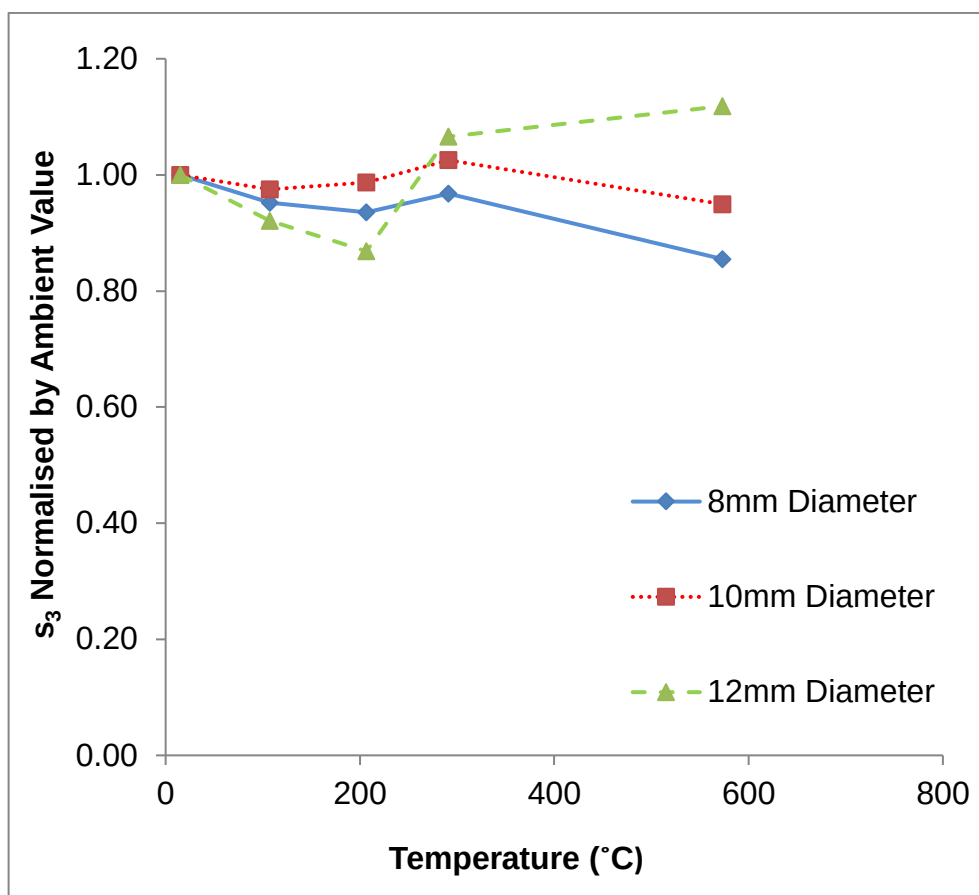


Figure C.4. Best-fit values of s_3 in fib model from regression analysis at different temperatures, normalised relative to value at 15°C

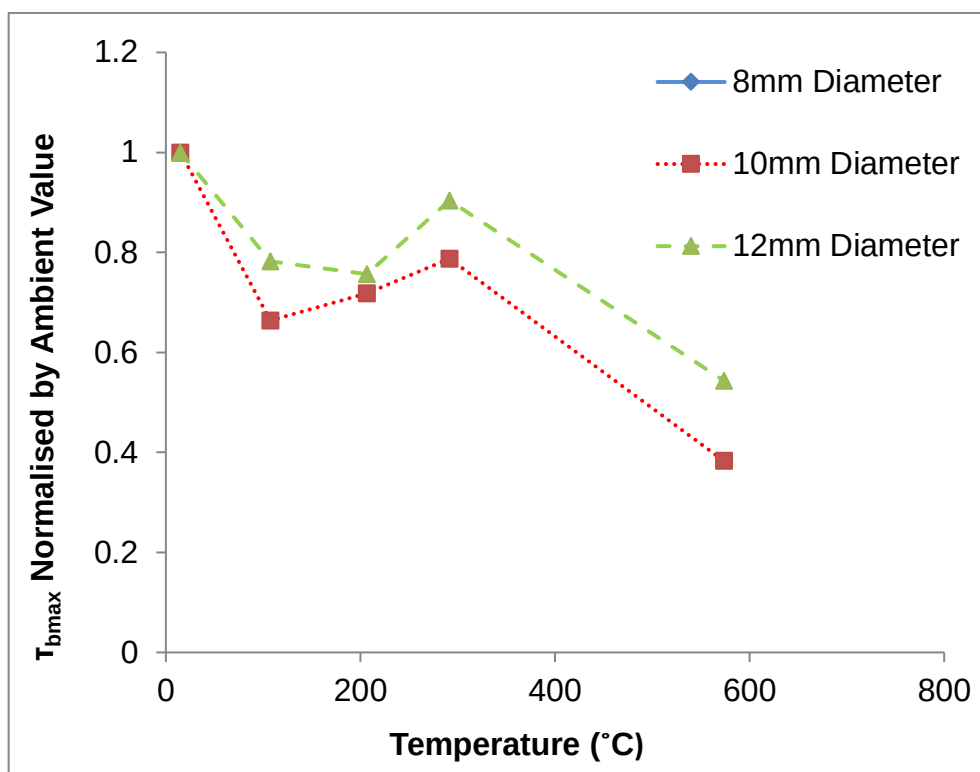


Figure C.5. Best-fit values of τ_{bmax} in fib model from regression analysis at different temperatures, normalised relative to value at 15°C

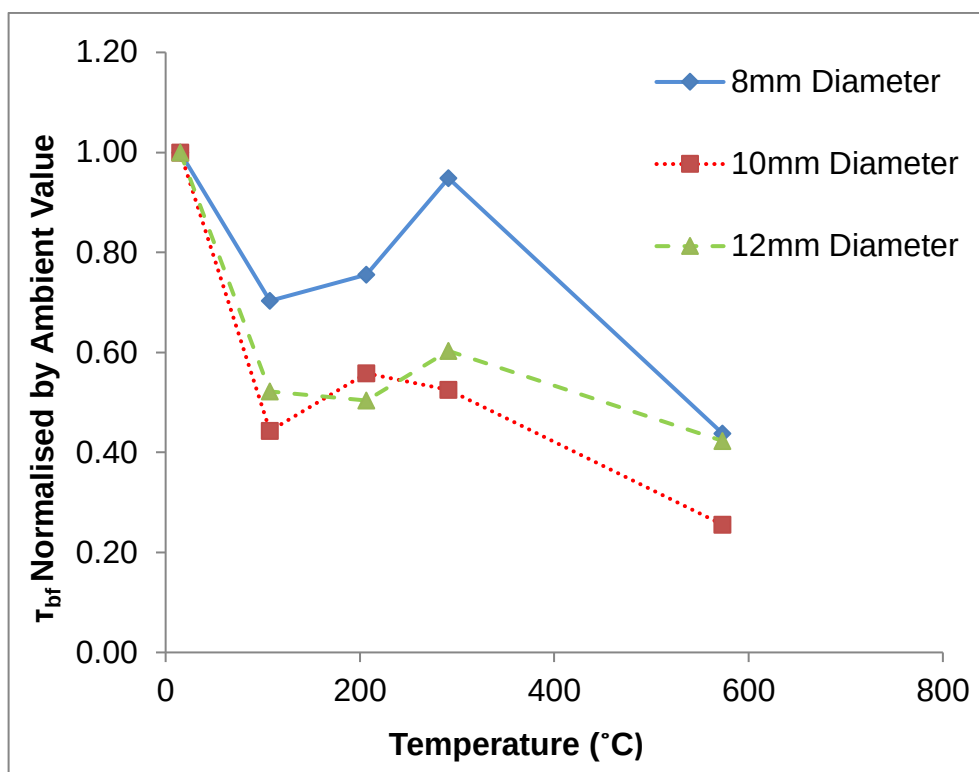


Figure C.6. Best-fit values of τ_{bf} in fib model from regression analysis at different temperatures, normalised relative to value at 15°C

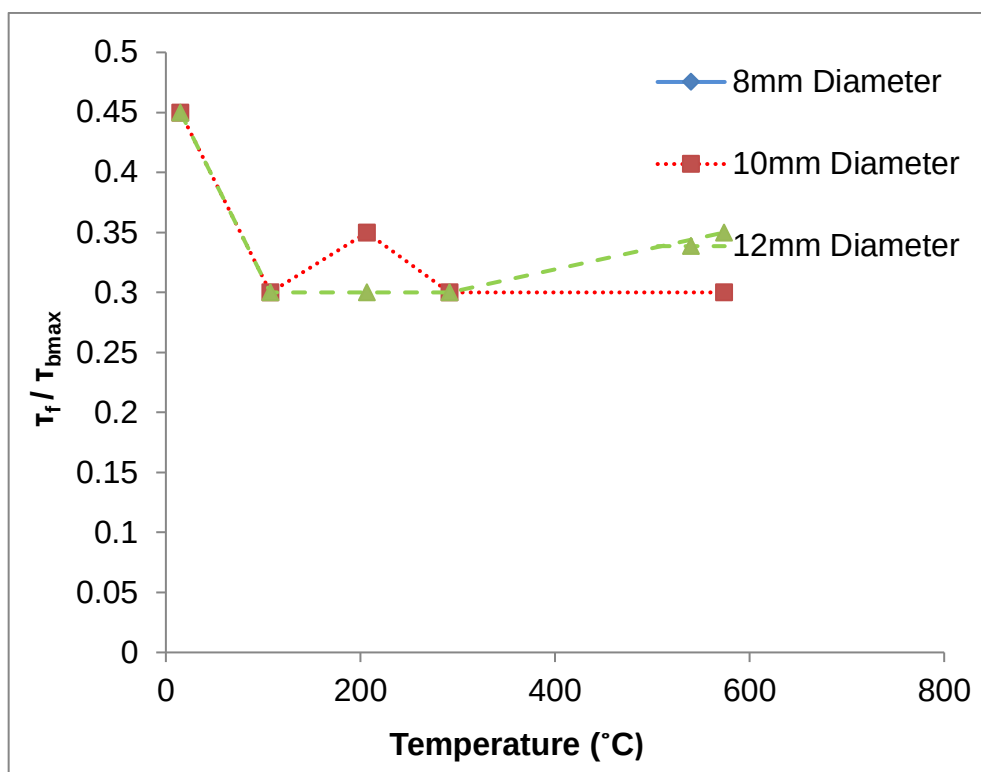


Figure C.7. Ratio of τ_f / τ_{bmax} best-fit values in fib model from regression analysis at different temperatures

Appendix D - Comparison between modified bond model and experimental results at elevated temperature

The figures here take the parameters s_1 , s_2 , s_3 and α at the values which had been identified from regression analysis as the best-fit values at the ambient temperature for each bar diameter. Equation (4.9) was used in the calculation of $\tau_{b\max,\theta}$ and τ_{bf} from the ambient experimental values of $\tau_{b\max}$. The figures in this appendix demonstrate the ability of the modified model to account for the effects of elevated temperatures. The quality of fit between the model and experimental data is weaker than the calculated results in appendix A because the figures in appendix D are estimated by the model for bars of different diameters and at different temperatures. These calculations also use compressive strength test results for a different concrete. The calculated stress-slip curves in appendix A were directly fitted to the experimental data which there are being compared with in the figures in appendix A and, therefore, have superior quality of fit.

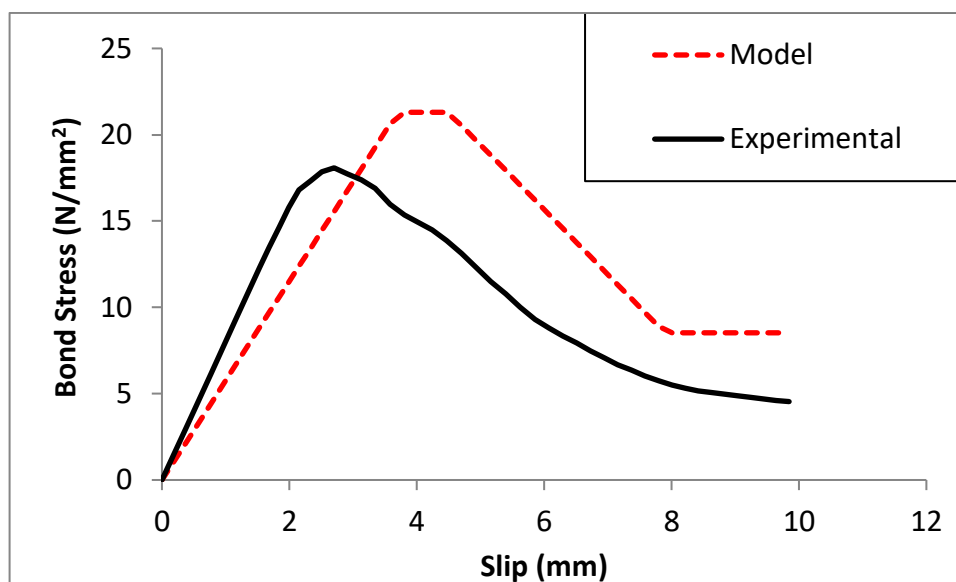


Figure D.1. Comparison of bond stress-slip of proposed model and experimental results for 10mm bar at 110°C

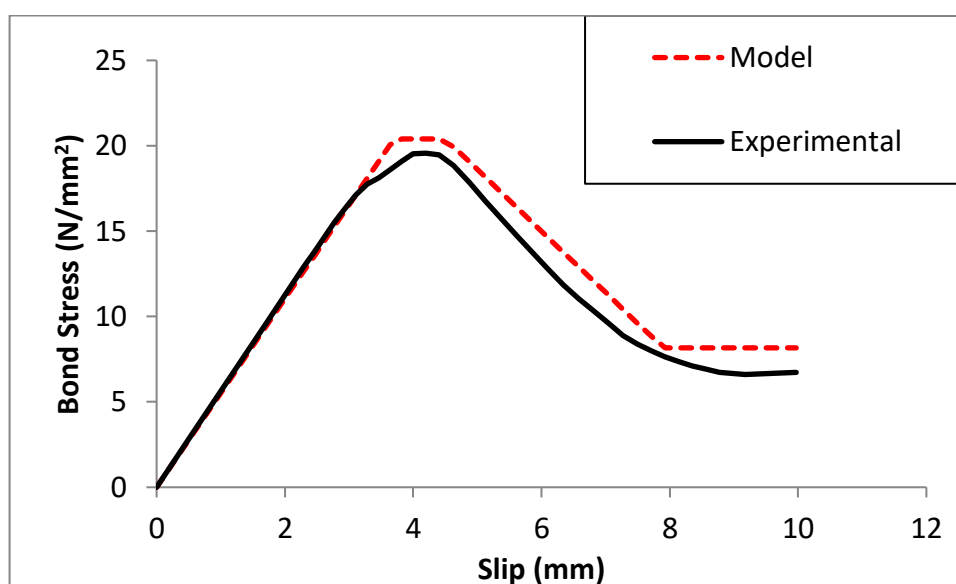


Figure D.2. Comparison of bond stress-slip of proposed model and experimental results for 10mm bar at 210°C

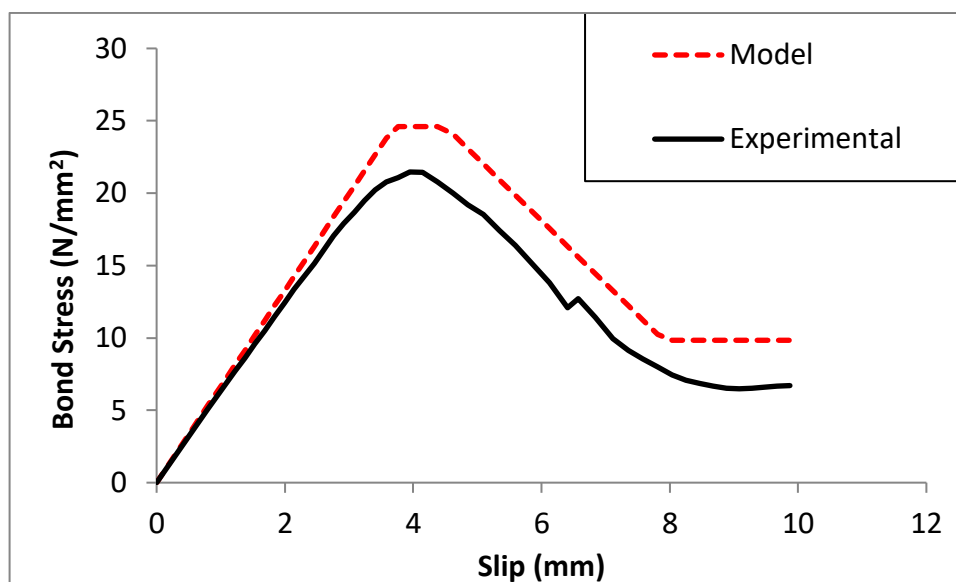


Figure D.3. Comparison of bond stress-slip of proposed model and experimental results for 10mm bar at 290°C

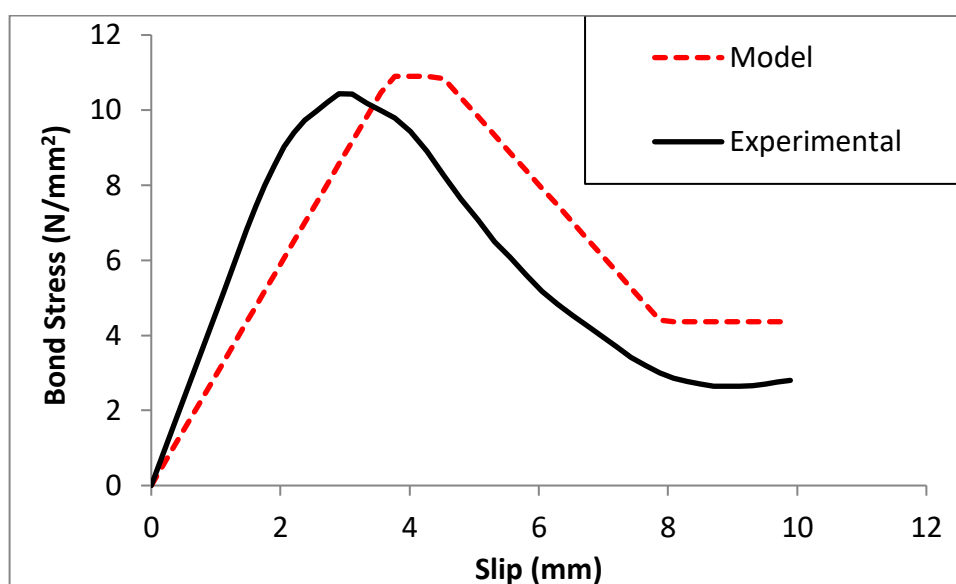


Figure D.4. Comparison of bond stress-slip of proposed model and experimental results for 10mm bar at 570°C

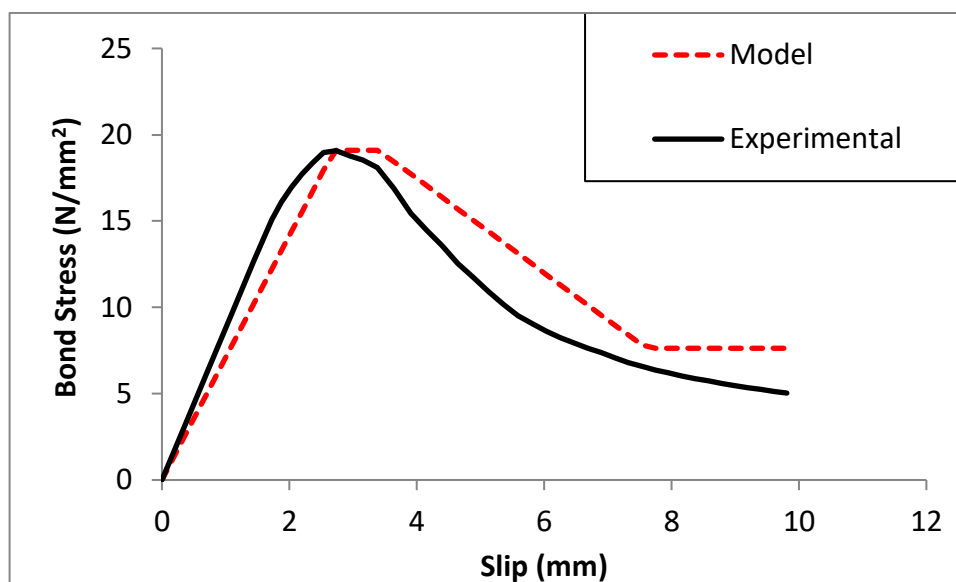


Figure D.5. Comparison of bond stress-slip of proposed model and experimental results for 12mm bar at 110°C

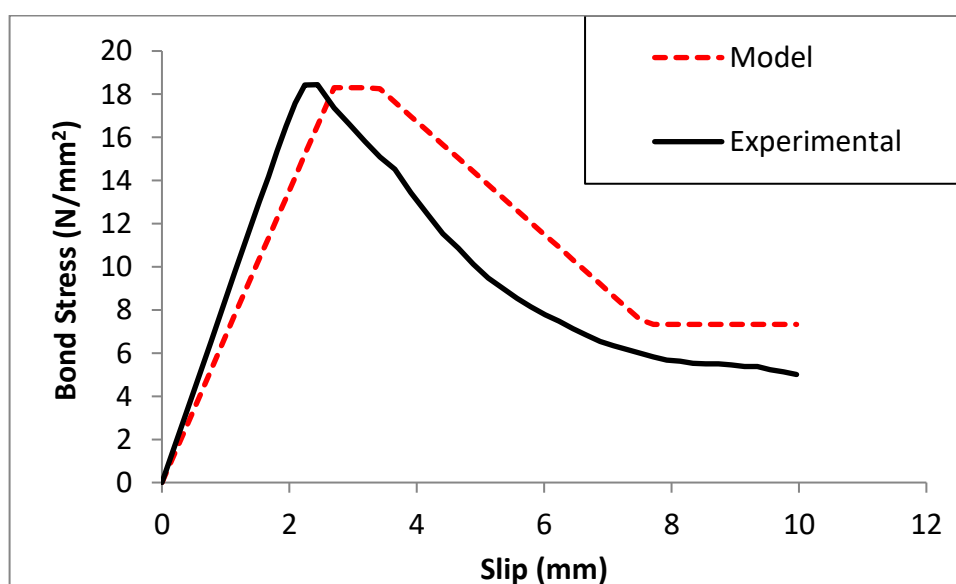


Figure D.6. Comparison of bond stress-slip of proposed model and experimental results for 12mm bar at 210°C

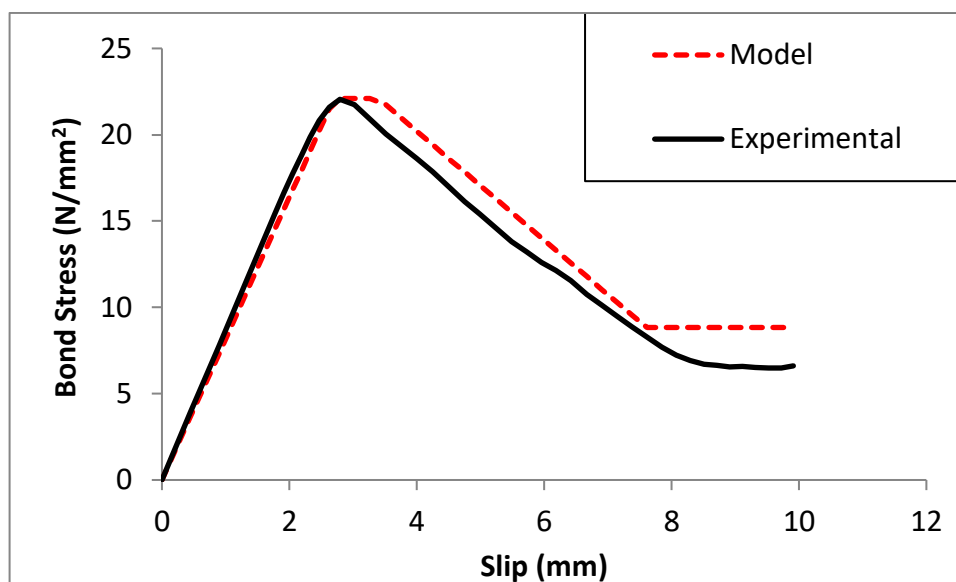


Figure D.7. Comparison of bond stress-slip of proposed model and experimental results for 12mm bar at 290°C

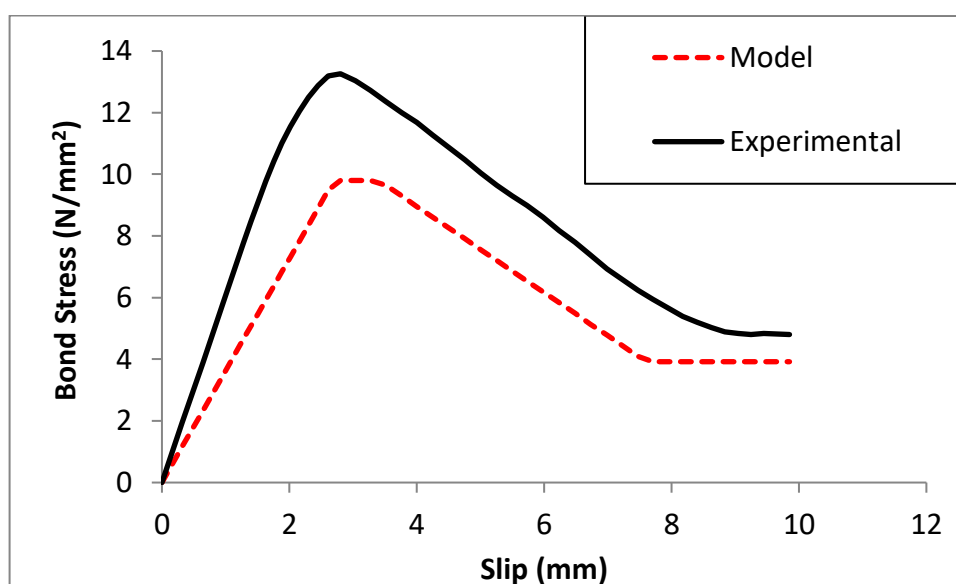


Figure D.8. Comparison of bond stress-slip of proposed model and experimental results for 12mm bar at 570°C

Appendix E - Experimental results

The figures in this appendix are the the raw, unprocessed, experimental pull-out force vs. displacement results for the experiments described in chapter 3.

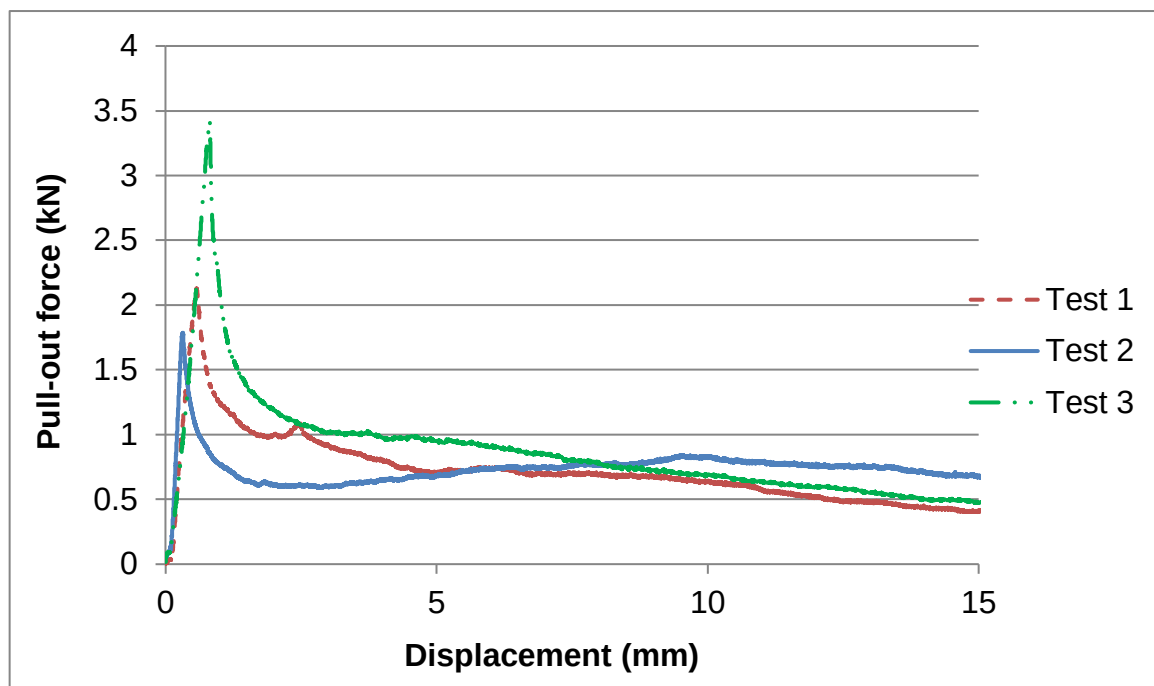


Figure E.1. Pull-out force vs. displacement for 8mm diameter plain round bar at 15°C

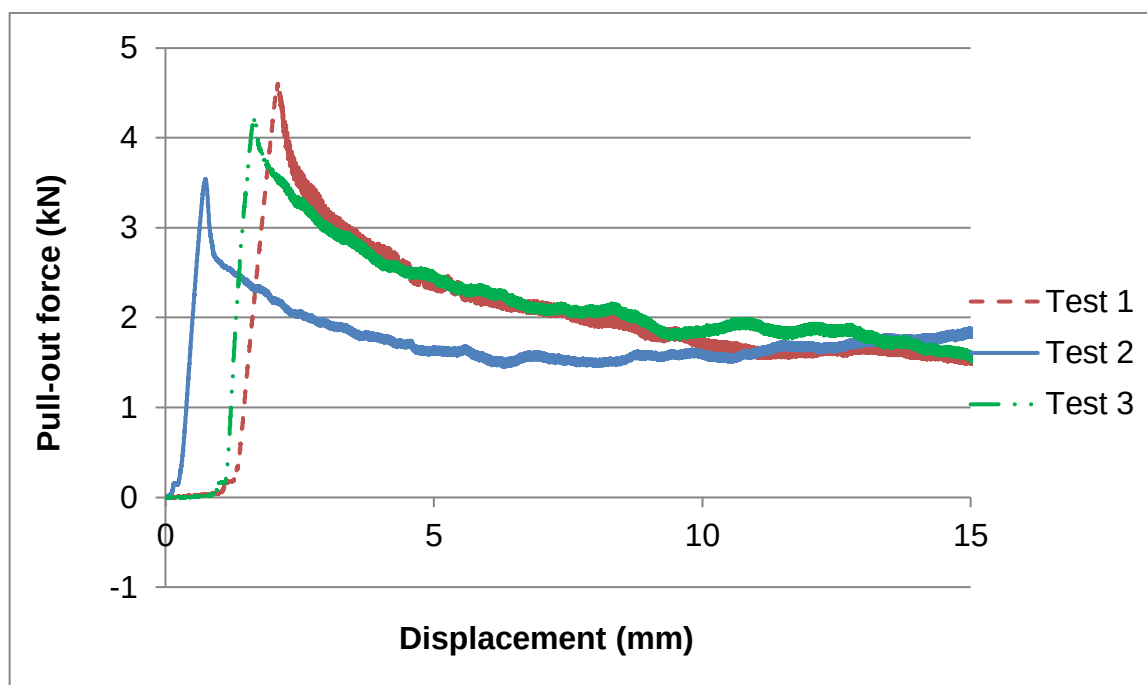


Figure E.2. Pull-out force vs. displacement for 8mm diameter plain round bar at
110°C

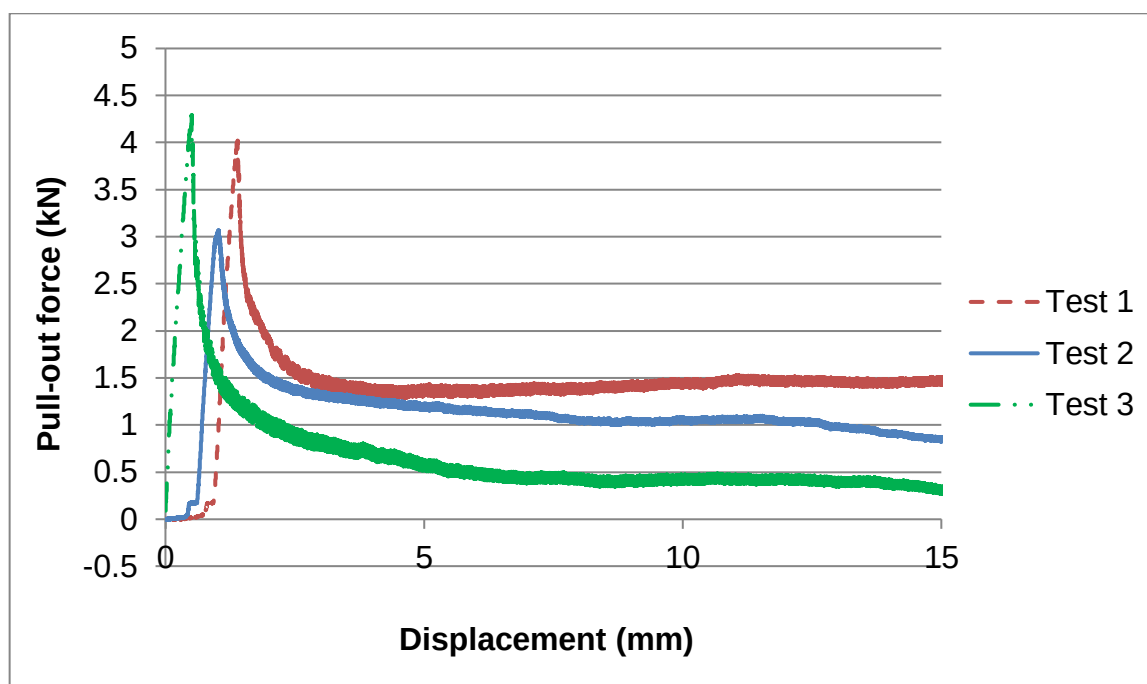


Figure E.3. Pull-out force vs. displacement for 8mm diameter plain round bar at
210°C

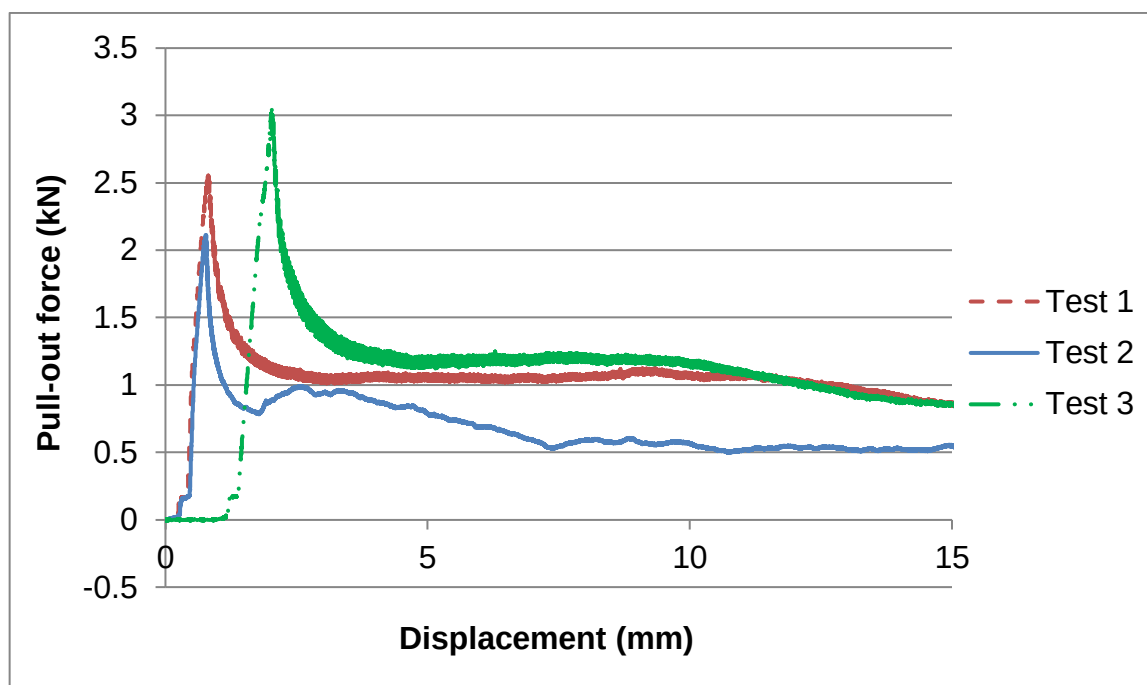


Figure E.4. Pull-out force vs. displacement for 8mm diameter plain round bar at
290°C

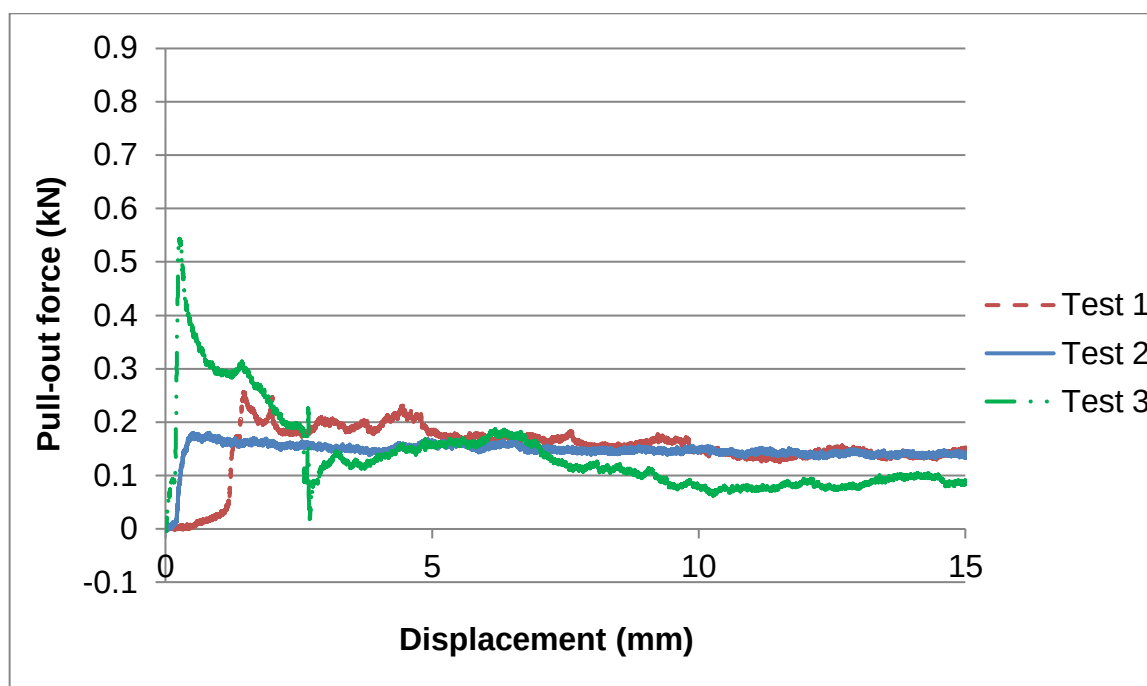


Figure E.5. Pull-out force vs. displacement for 8mm diameter plain round bar at
570°C

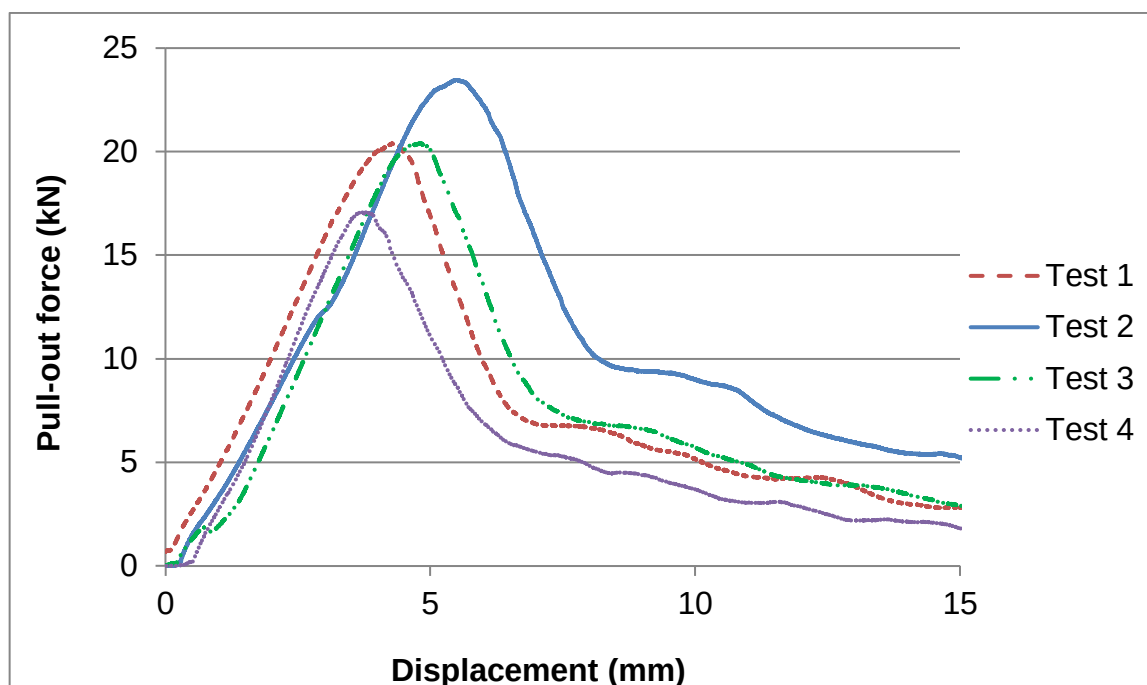


Figure E.6. Pull-out force vs. displacement for 8mm diameter deformed bar at 15°C

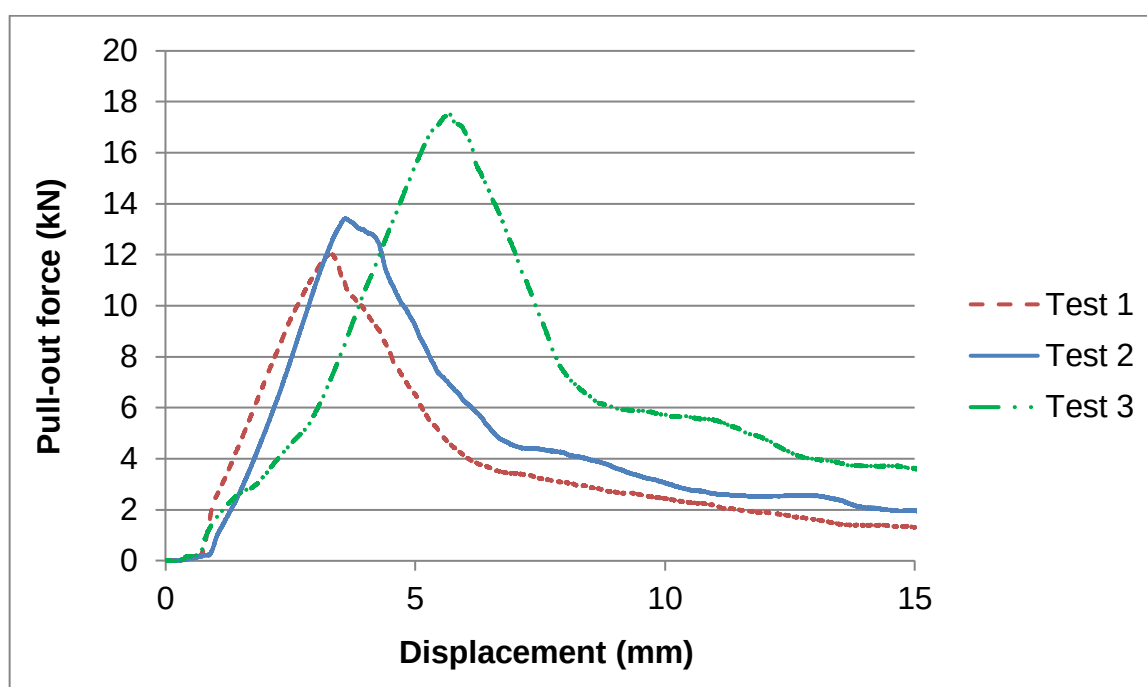


Figure E.7. Pull-out force vs. displacement for 8mm diameter deformed bar at 110°C

Test 2 in figure E.8 is thought to have had an unintentional pre-load. Given the error in test procedure, there is some uncertainty over this result.

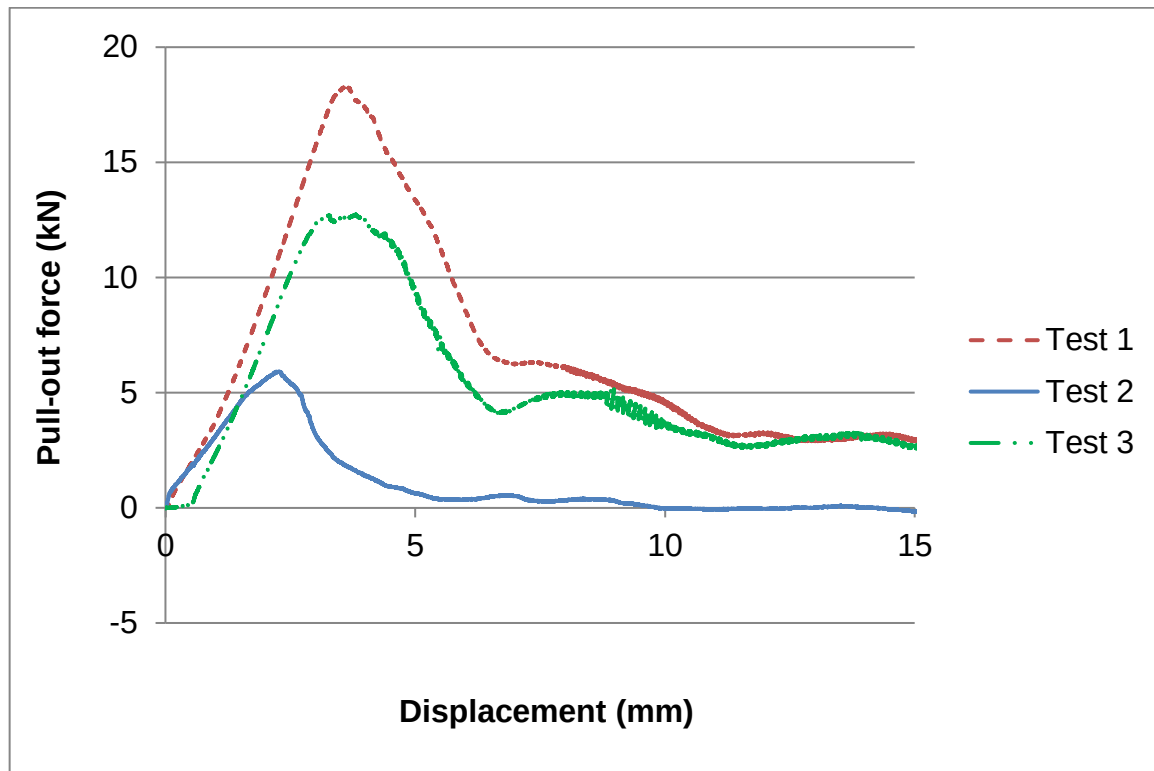


Figure E.8. Pull-out force vs. displacement for 8mm diameter deformed bar at 210°C

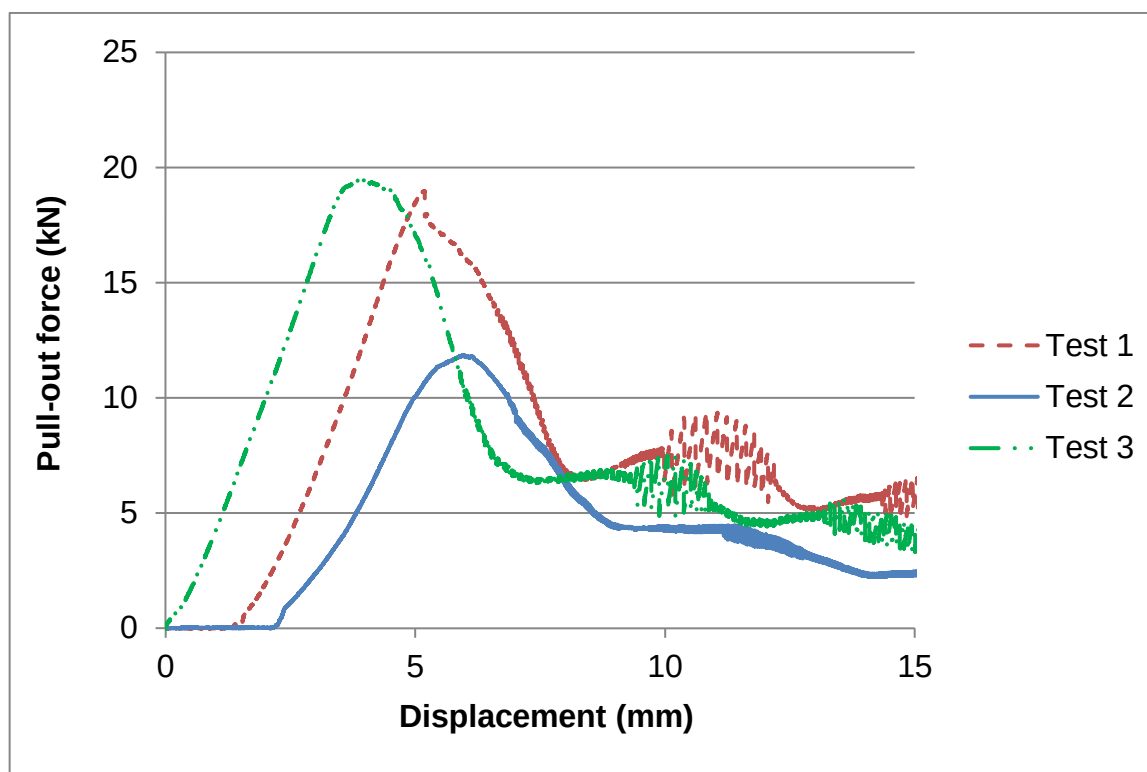


Figure E.9. Pull-out force vs. displacement for 8mm diameter deformed bar at 290°C
(unknown source of noise at large displacements)

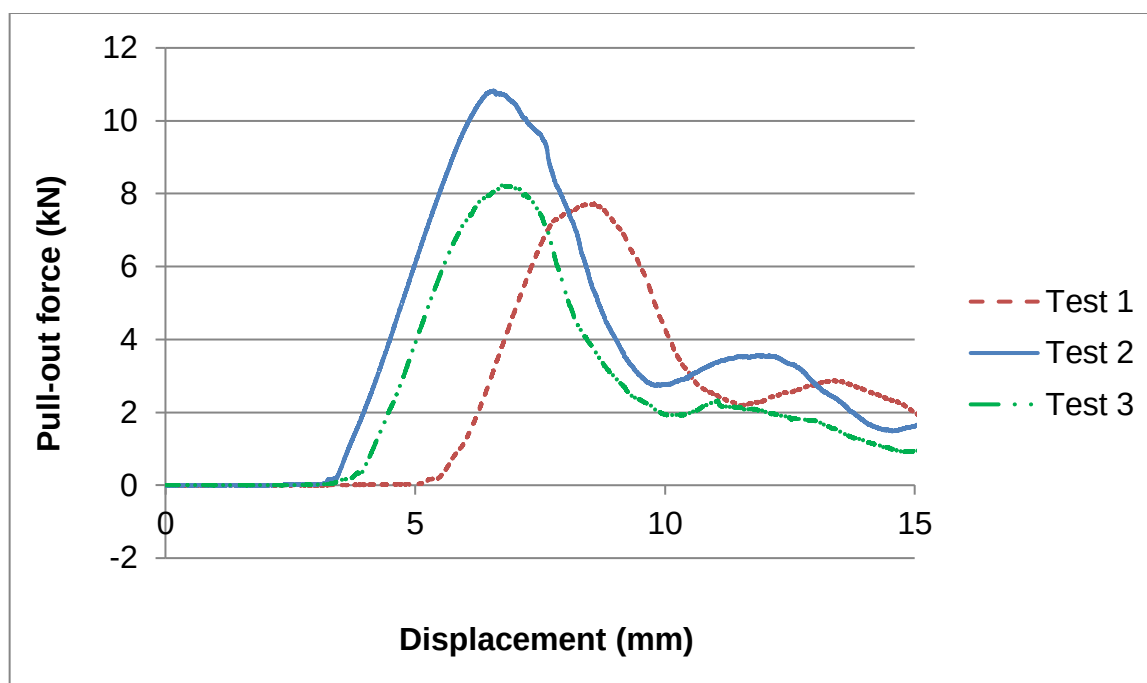


Figure E.10. Pull-out force vs. displacement for 8mm diameter deformed bar at
570°C

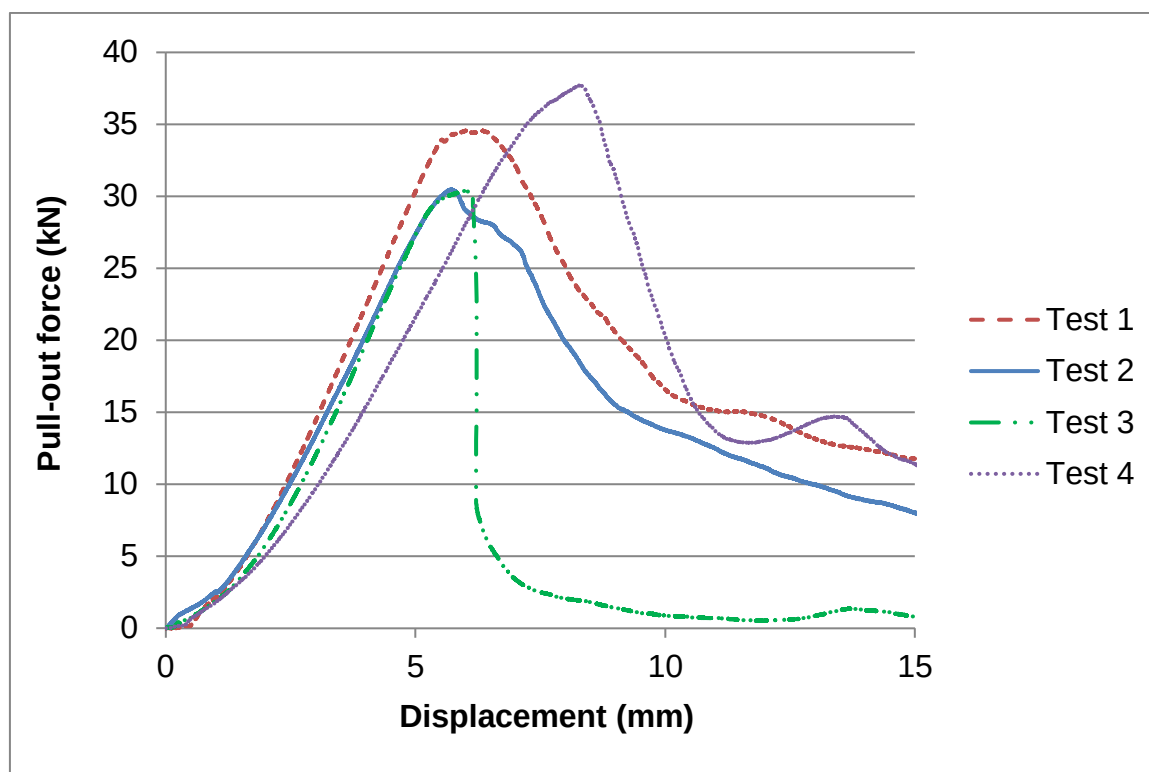


Figure E.11. Pull-out force vs. displacement for 10mm diameter deformed bar at 15°C (test 3 - splitting failure)

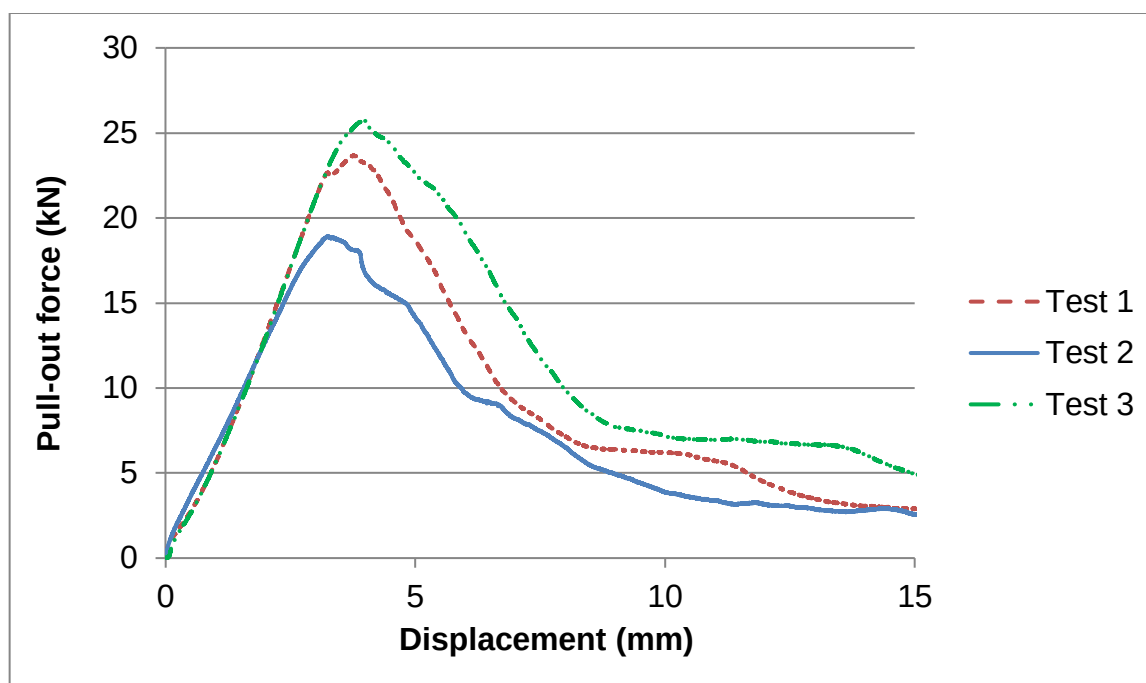


Figure E.12. Pull-out force vs. displacement for 10mm diameter deformed bar at 110°C

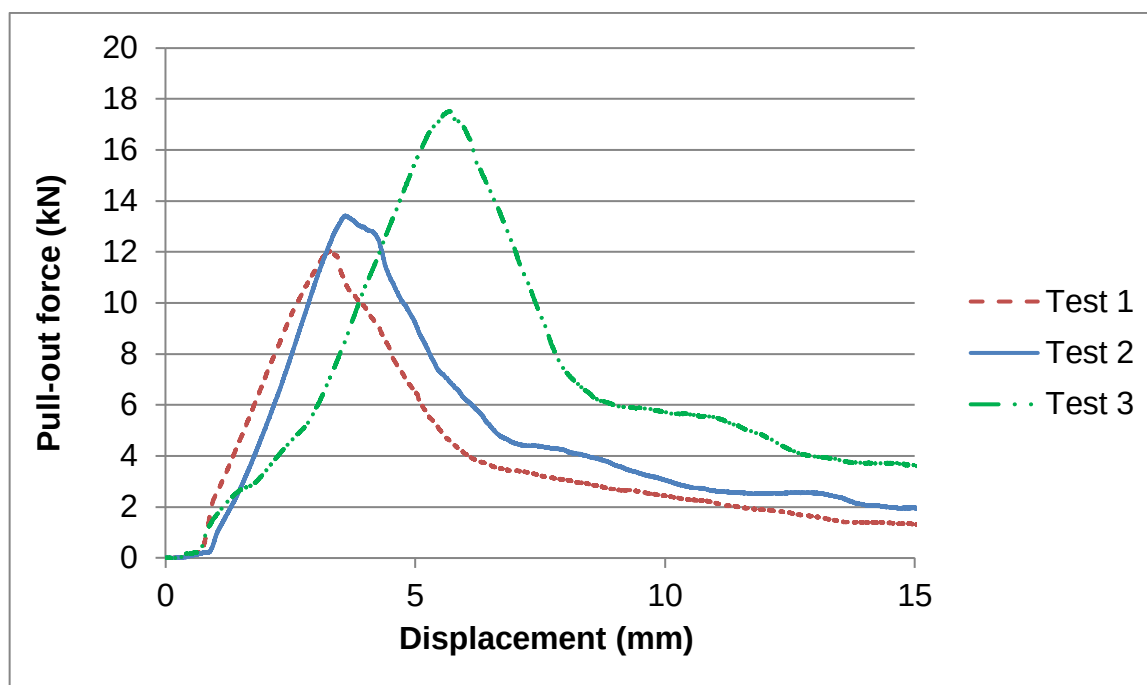


Figure E.13. Pull-out force vs. displacement for 10mm diameter deformed bar at
210°C

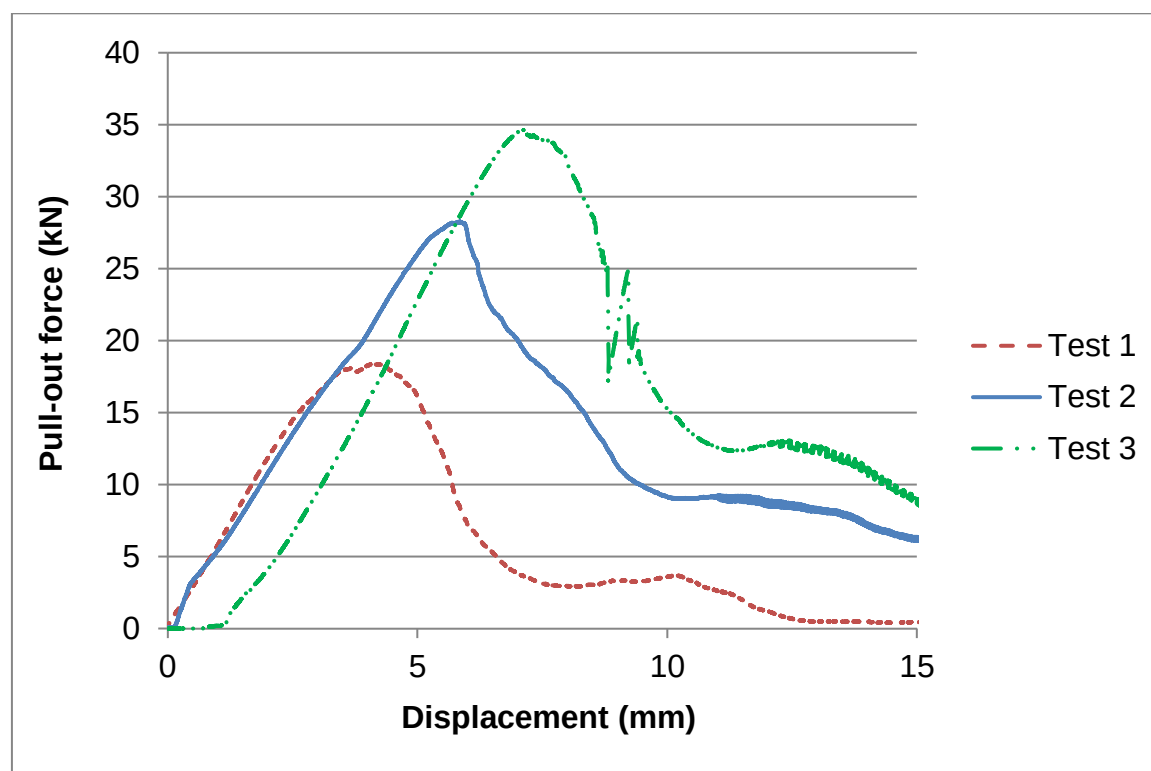


Figure E.14. Pull-out force vs. displacement for 10mm diameter deformed bar at
290°C

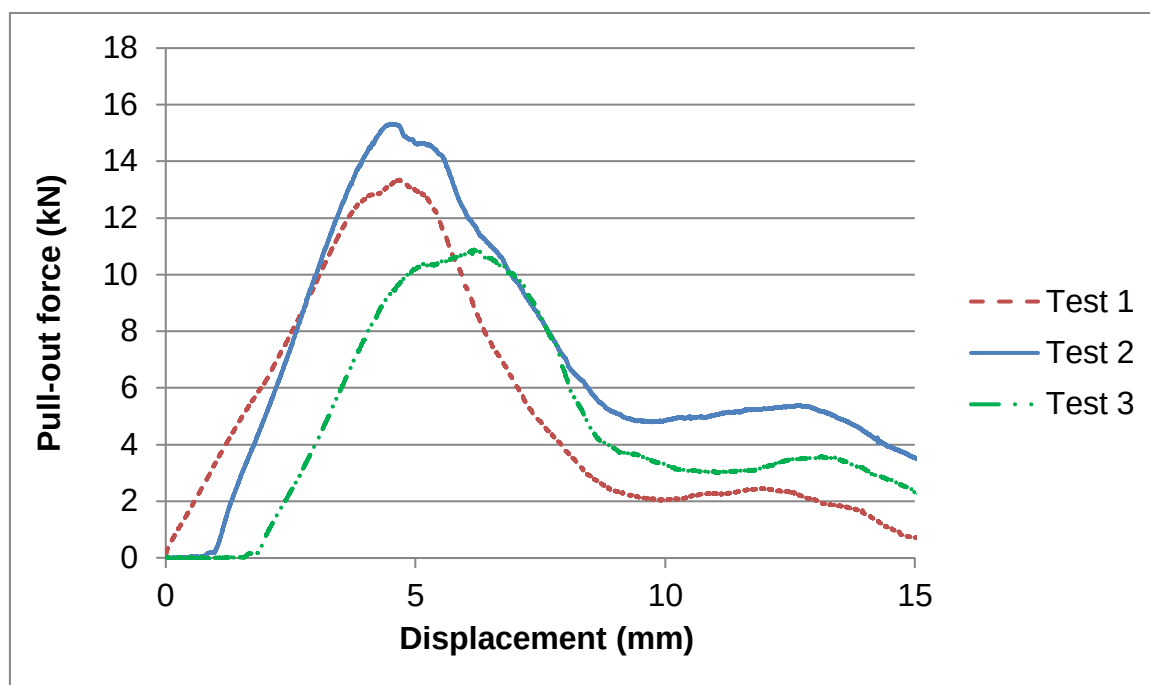


Figure E.15. Pull-out force vs. displacement for 10mm diameter deformed bar at
570°C

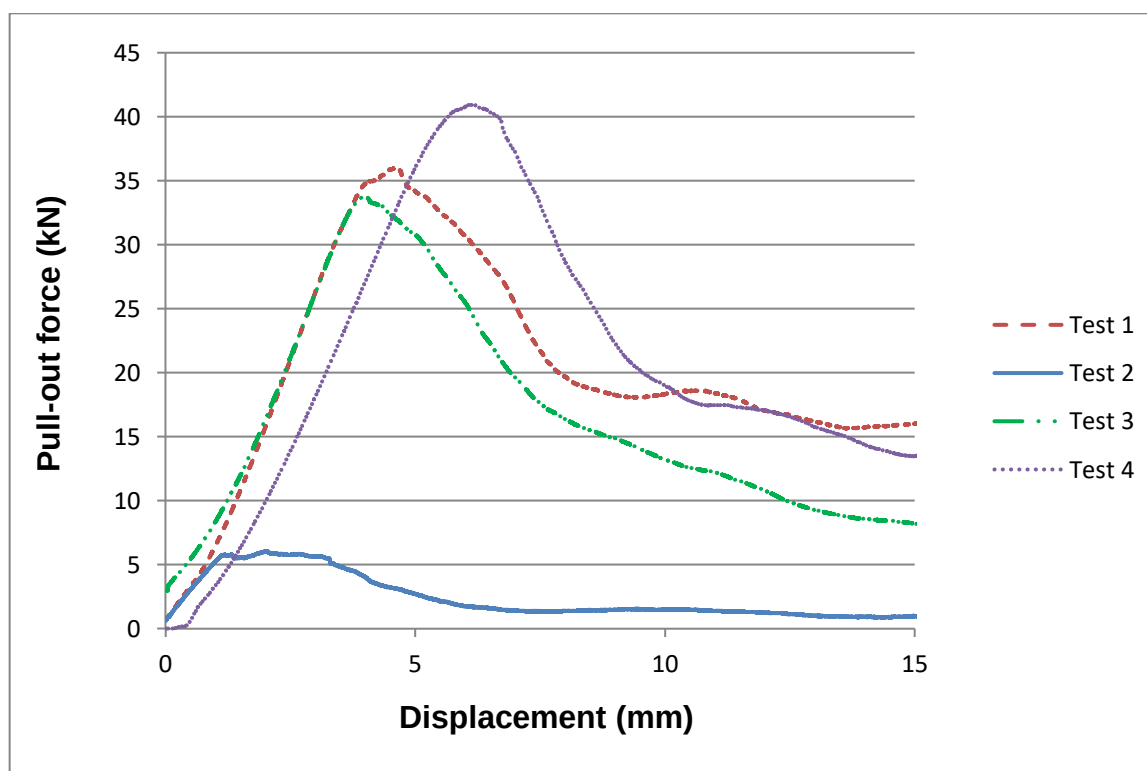


Figure E.16. Pull-out force vs. displacement for 12mm diameter deformed bar at
15°C

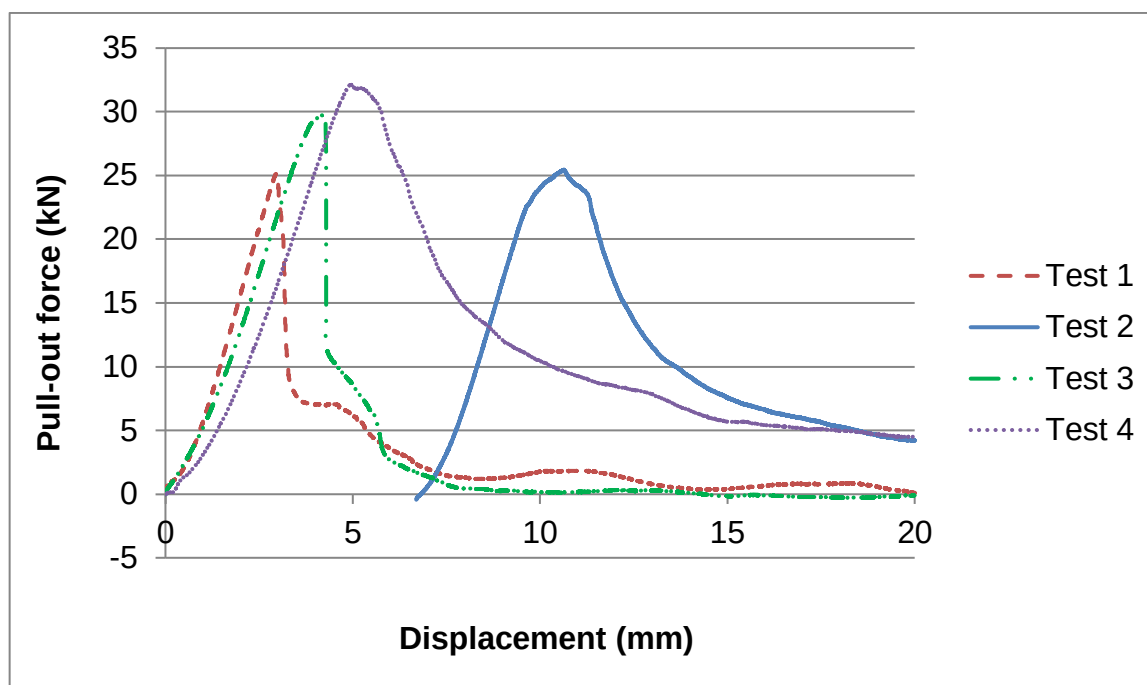


Figure E.17. Pull-out force vs. displacement for 12mm diameter deformed bar at 110°C

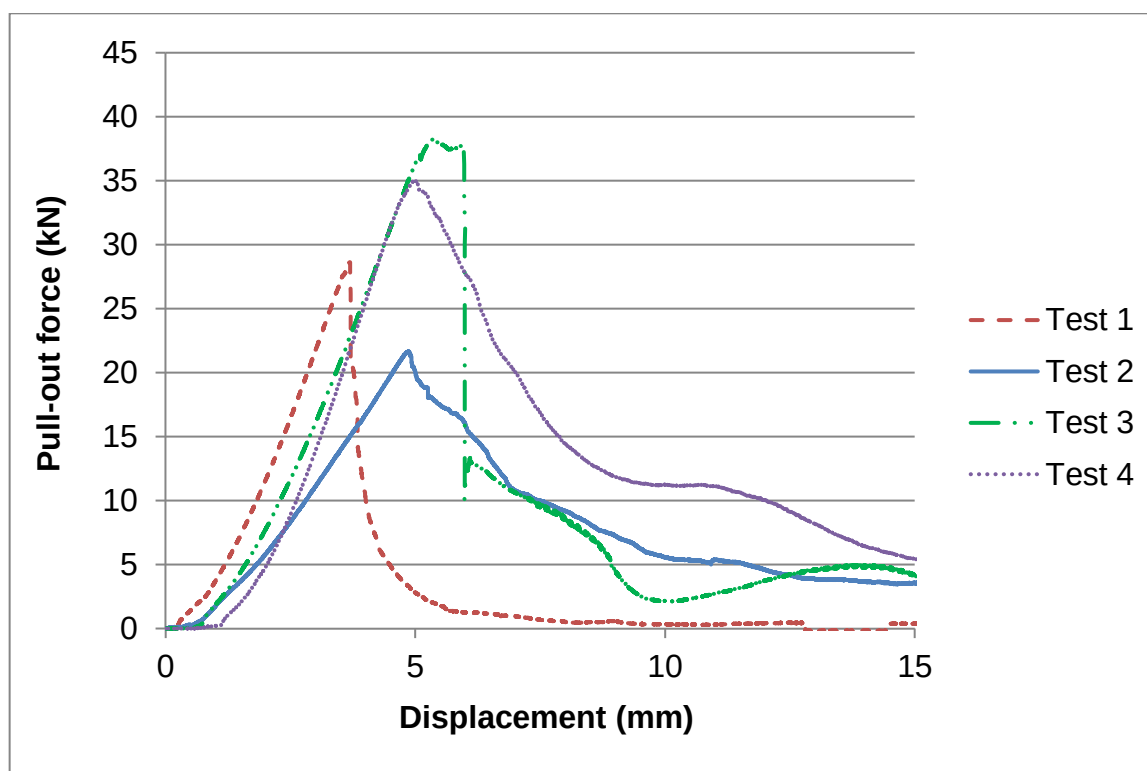


Figure E.18. Pull-out force vs. displacement for 12mm diameter deformed bar at 210°C (tests 1 and 3 - splitting failures)

Test 2 in figure E.19 is thought to have had an unintentional pre-load. Given the error in test procedure, there is some uncertainty over this result.

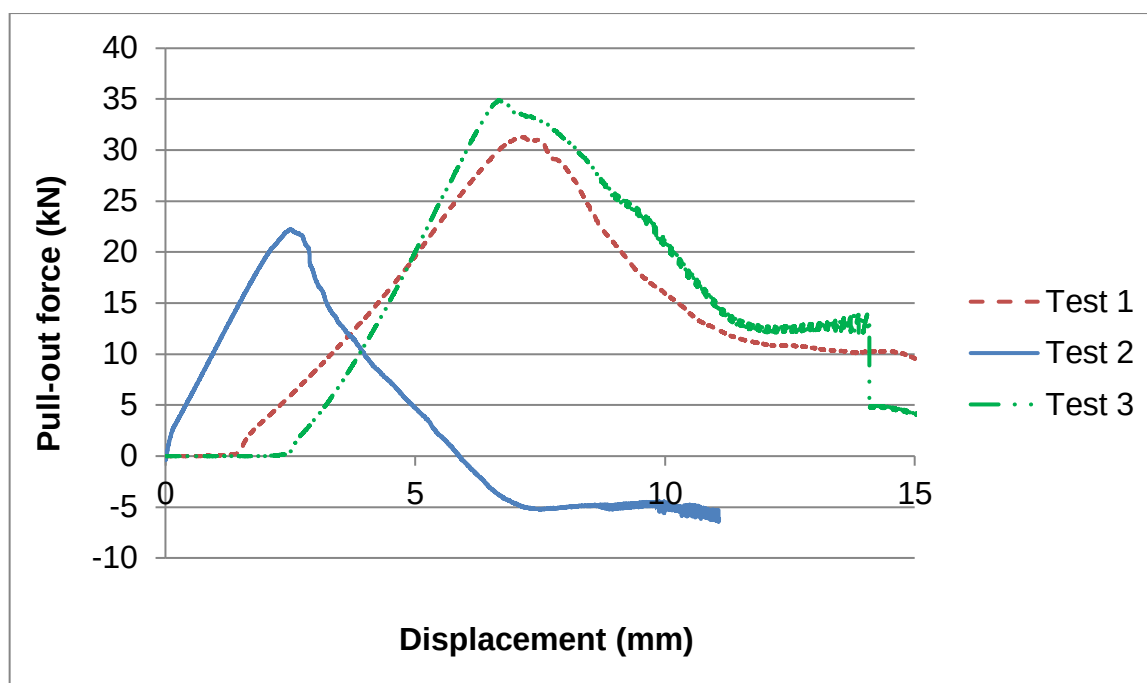


Figure E.19. Pull-out force vs. displacement for 12mm diameter deformed bar at 290°C

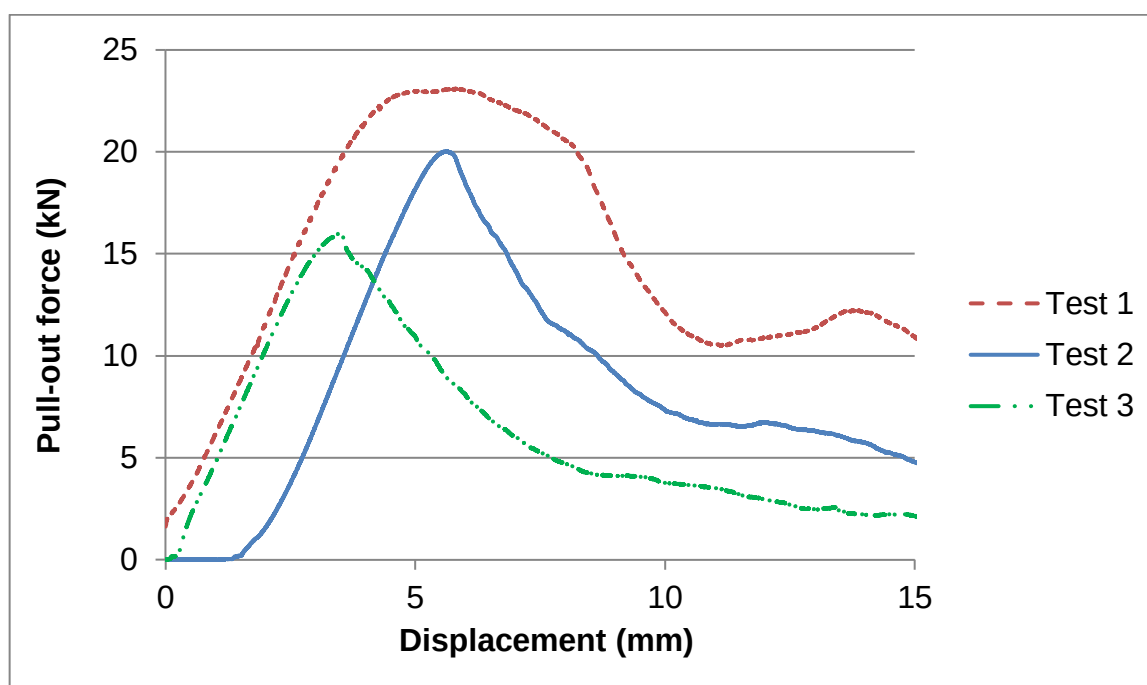


Figure E.20. Pull-out force vs. displacement for 12mm diameter deformed bar at 570°C