The Structural Response of Reinforced Concrete Columns During and After Exposure to Non-Uniform Heating and Cooling Regimes

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The Structural Response of Reinforced Concrete Columns During and After Exposure to Non-Uniform Heating and Cooling Regimes

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Declaration

This thesis and the work described within have been completed solely by Jamie Maclean at the BRE Centre for Fire Safety Engineering at the University of Edinburgh, under the supervision of Prof Luke Bisby. Where others have contributed, or sources are quoted, references have been provided.

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Abstract

In addition to the immediate life safety concerns during building fires, uncontrolled fires within buildings have the potential to cause extensive structural damage. Current design guidance for structures in fire focuses exclusively on the life safety of the occupants within buildings. With regard to the structure this is generally achieved by specifying a defined fire resistance period during which structural integrity must be maintained and fire spread must be prevented. This is to ensure that the building's egress routes are not compromised until all occupants have escaped from the building and fire-fighting operations have been completed. Designers are not typically required to explicitly consider the residual post-fire effects on structures. Particularly in relation to concrete structures which tend to perform well in fire and can often be reinstated, this raises questions about whether the post-fire effects are important from a life safety perspective.

This thesis explores the applicability of some of some simple models to reflect the complex behaviour observed when symmetrically reinforced concrete columns are subjected to non-uniform heating regimes. An experimental test series was designed to provide an extensive data bank of the performance of a single reinforced concrete column subjected to many combinations of different loading and heating conditions. To this end 46 geometrically identical reinforced concrete columns were subjected to a combination of loading and heating conditions and both the thermal and mechanical response were monitored during both

the heating and cooling phases of the experiments. All surviving columns were destructively tested to determine their residual performance 24 hours after cooling back to ambient temperature conditions. A sectional analysis model is presented to determine the load-moment interaction of a reinforced concrete column after exposure to elevated temperatures. This has been aided with the use of non-destructive testing of each of the columns with the aim of helping practitioners determine the post-fire properties and to aid in the residual analysis of a concrete structure exposed to elevated temperatures.

In comparing the results to the current design guidance available it would be expected that each experiment would react in an identical fashion as the design of each of the columns and the external heat source is identical in each case. This experimental test series has concluded that this is not necessary the case. From a post-fire residual structural performance standpoint, this raises a number of questions regarding how practitioners can approach the assessment and analysis of reinforced concrete structures in the future.

Given the findings of this work, and the one of a kind data bank that has been created as regards to the performance of reinforced concrete columns subjected to non-uniform heating and cooling regimes, fire engineering modellers now have the capability to validate Finite Element models against a series of 46 reinforced concrete columns through the full range of heating, cooling and post-cooling structural performance. The results of which have profound implications for the current design methodologies recommended. This data bank can now be used to validate such models and inform future design methodologies.

Lay Summary

Concrete is one of the most popular construction materials used in the modern built environment. It is therefore important to understand the material behaviour and structural response of concrete when subjected to all kinds of external influences such as wind loads, earthquakes and fire, etc. This thesis describes the experimental methods employed to investigate, given the current breadth of knowledge of the material properties and response of concrete in fire, whether it is currently possible to accurately predict the behaviour of a symmetrically reinforced square concrete column subjected to high temperatures in the middle third of the element from when the load is initially applied through both the heating and cooling phase of the experiment.

Forty-six geometrically identical reinforced concrete columns have been designed, cast and tested in an experimental test series and the results have been compared to the current design guidance used in the construction of this type of structural element. The results of this experimental test series have uncovered new and interesting behaviour of concrete that is not explicitly considered in design. In addition to identifying these gaps in the current design guidance, a large data bank has been collated which can be used by engineers to better understand the "real" behaviour of reinforced concrete columns subjected to elevated temperatures to fill the "gaps" identified as part of this thesis.

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Chapter 1

Introduction

1.1 Motivation for Research

Reinforced concrete is one of the most widely utilised building materials. It is heavily used in the construction of buildings, bridges, dams, roads and almost every other form of civil infrastructure. In recent years, as innovation and optimisation has resulted in the design of larger, more complex structures, there has been considerable investment in developing new types of concrete and researching their structural properties under different relevant conditions. It is so heavily used in the construction industry that it is estimated to account for 4-5% of the total worldwide CO_2 emissions [7]. This trend does not show any signs of slowing in the coming years and concrete structures, many of which will necessarily involve structural optimisation and novel forms of high strength and high performance concrete so as to attempt to reduce the buildings carbon footprint, will only increase in both number and complexity. Alongside this, it is estimated that approximately 14,500 fires occurred per year between 2009 and 2013 in The United States alone [8]. The severity and resulting cost (both direct and indirect) varied greatly amongst these fires; however it is worth remembering that, given societies reliance on reinforced concrete in the built environment and the number of fire incidents that occur, understanding the response of reinforced concrete structural elements and structural systems when exposed to elevated temperatures (via fire) is crucial. There are, of course, many different types of reinforced concrete construction and structural elements i.e. slabs, beams, columns, pre-tensioned, post-tensioned, etc., each of which has specific loading conditions and restraint continuity configurations warranting investigation in fire. It is outwith the scope of this thesis to examine and discuss all types of reinforced concrete construction. However, given the importance of vertical load bearing structural elements for assuring life safety in building fires, and considering the ongoing innovation and optimization of both material properties and structural designs of concrete columns in modern buildings, the current thesis looks specifically at reinforced concrete columns. This thesis discusses, and then addresses the current gaps in knowledge with regard to the performance (i.e. thermal and mechanical response) of reinforced concrete columns before, during, and after exposure to elevated temperatures (as would be expected in an unwanted building fire).

In the event of an uncontrolled fire within a building, or any other situation where a concrete structure is exposed to severely elevated temperatures, a number of different stages hold significance - in different ways - in terms of the thermal and mechanical response of the elements involved. These stages have been detailed graphically in Figure 1.1 with a view to setting the overall stage for the research presented in this thesis.

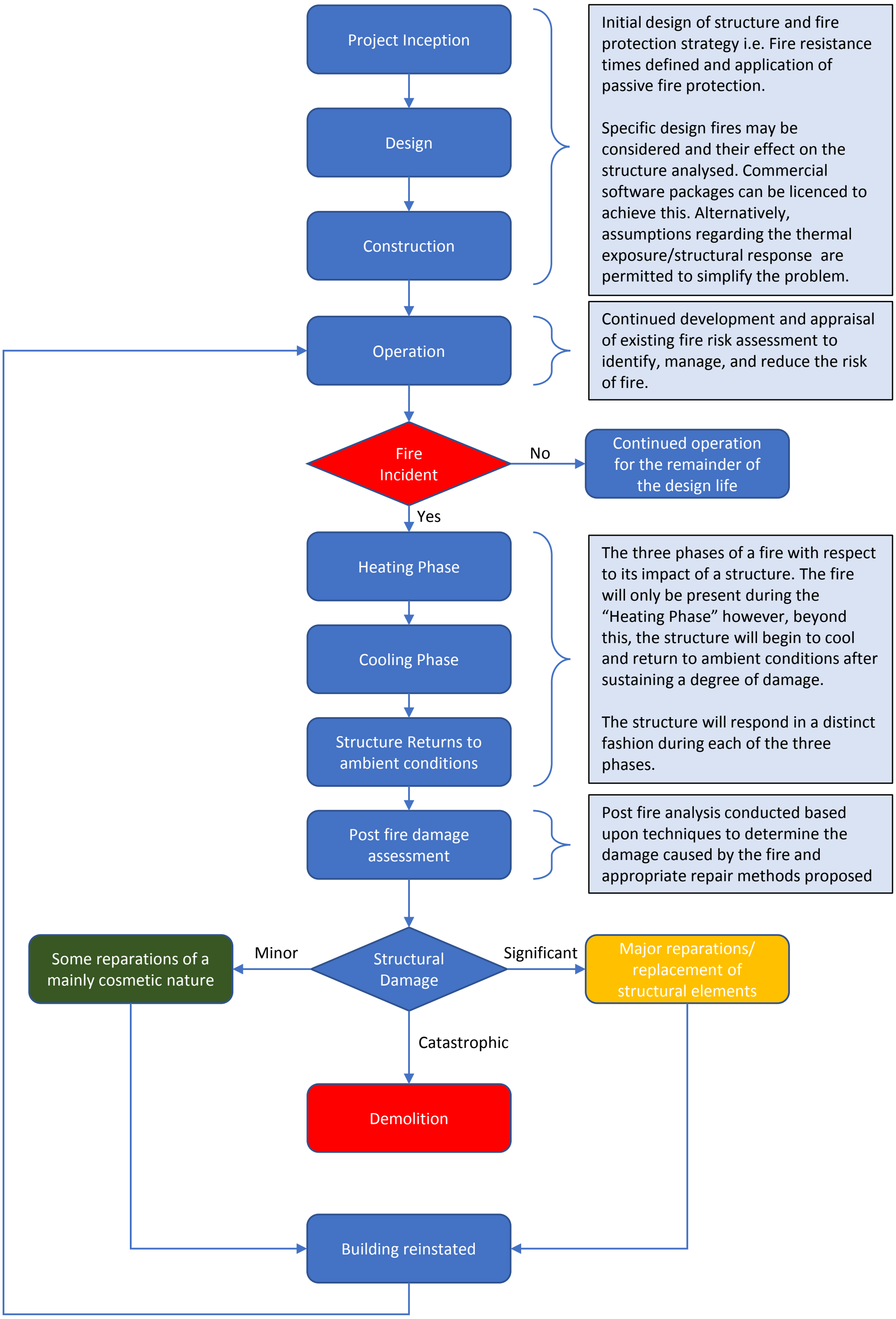


Figure 1.1: Stages of a fire as seen by a reinforced concrete structure (or structural element)

As detailed in Figure 1.1, there are three main fire-related stages that require consideration in the design of any structure exposed to elevated temperatures: (1) the heating phase, (2) the cooling phase, and (3) the residual phase (when the elements have cooled back to ambient temperature conditions). Within current design guidance [4] internationally the primary (or indeed, only real) consideration is given to the initial heating phase, with little or no serious consideration given to either the cooling phase or the residual (i.e. post fire) phases beyond a fire incident itself. This is primarily a result of the clear focus on the life safety objectives within; building regulations, design codes, compliance testing procedures, and guidance documents used in the design and approval of reinforced concrete buildings for fire safety. The inherent assumption within the current design procedures used internationally is that, as long as the building maintains structural integrity for a defined period of time of exposure to a standard temperature versus time curve (when isolated structural elements are tested within a standard fire resistance testing furnace) for people to escape, and for fire fighting operations to take place, what happens beyond this point is, for all intents and purposes, irrelevant.

The above approach can of course be brought into question when, as previously discussed, buildings are becoming larger, more optimised, and more complex, resulting in differing structural response to fire combined with (in some cases) longer evacuation times. Evacuation times which, with the spread and localised burnout of fire within a building, may well result in certain elements entering the cooling phase while occupants/fire fighters are still present within the building, possibly leading to localised or even global structural collapse with resulting loss of life and property.

Beyond the initial life safety concerns within a building, in the event of a fire, the structure will increase in temperature which, after cooling, may result in substantially different material properties from that of undamaged concrete at ambient temperature [9]. Concrete has historically proven to behave favourably in this area as a result of its low thermal conductivity and high heat capacity, which means heat penetration into concrete elements takes exposure to high temperatures for an extended period of time. This behaviour is favourable as compared with other materials such as steel, however the process of determining the “damage” caused by exposure to elevated temperatures is much more difficult for concrete than steel, specifically because of the steep thermal gradients generated with concrete during heating and cooling. As a result of the slow propagation of the thermal wave in the concrete and the permanent loss of strength and stiffness of the concrete at relatively low temperatures (e.g. 300-500^{o}C) [9], accurate prediction of the post-heating residual properties of the concrete has proven to be challenging [10]. This thesis aims to shed light on this area, as well as in the area of the structural fire response of non-uniformly heated concrete columns, and to highlight any remaining gaps in knowledge that need to be explored in order to accurately predict the post-fire behaviour of reinforced concrete columns.

1.2 Research Objectives

The aim of this thesis is to investigate the structural response of reinforced concrete columns before, during, and after exposure to elevated temperatures. Specifically, this research project looks into the effect of localised, non-standard heating of reinforced concrete columns under sustained load, and aims to interrogate the current design methods used to determine their validity in such non-standard configurations. In addition to studying how reinforced concrete columns are designed and analysed to assure adequate structural response to fire, this project aims to improve the general understanding of applications of some specific post-fire non-destructive testing techniques, which may be used to determine the damage sustained by the concrete columns as a result of exposure to elevated temperatures such as would likely be the case during a building fire. In order to achieve this, the primary objectives of the research project presented in this thesis are:

- to experimentally investigate and computationally/analytically predict the ambient temperature response of the reinforced concrete columns under varying conditions of eccentric axial loading up to failure;
- to experimentally investigate the effect of severe localised heating on the mechanical performance of reinforced concrete columns under a number of different sustained eccentric axial loading/heating conditions, the results of which may be used to validate the computational/analytical methods used to predict the response of these columns;
- to experimentally investigate various non-destructive testing techniques which may be used in the post-fire residual property assessment of reinforced concrete structures to determine the level of "damage" sustained during severe heating;
- to take steps towards quantifying and predicting the response of the reinforced concrete columns before, during and after exposure to elevated temperatures through the development of an analytical predictive model; and
- to provide guidance to structural designers on the fire performance and post-fire residual structural response of locally heated reinforced concrete columns under sustained eccentric axial compressive load.

In achieving these objectives, it is then possible to determine where gaps in the current knowledge remain in the understanding of the mechanical behaviour of reinforced concrete columns subjected to elevated temperatures. Recommendations for the appropriate next steps are then identified, where they lie outside the scope of the current research project.

1.3 Scope of this Research Project

As previously stated, this project aims to investigate the performance of reinforced concrete columns subjected to non-standard but severe, localised heating conditions. To fulfil this aim, an experimental program has been conducted which consists of tests on 46 geometrically identical reinforced concrete columns. These columns have been tested in a non-standard, bespoke testing set up to investigate the influence of a number of relevant parameters on their performance before, during, and after exposure to elevated temperatures.

All of the columns tested were geometrically identical and only two concrete mixes were used. It is therefore not intended that the results of these experiments be extrapolated to every possible scenario when it comes to design, rather it is intended that this work interrogate and challenge the current state of the art in reinforced concrete structural fire analysis. Given the current knowledge of material properties and the composite behaviour of reinforced concrete structures, it is worth asking whether it is possible to accurately predict the behaviour of this one relatively "simple" reinforced column subjected to numerous different, carefully controlled and systematically varied, loading and heating scenarios.

Therefore, in addition to undertaking the experimental program described, the results have been compared and contrasted against the currently available design guidance, where appropriate, to determine its validity in real world scenarios and to suggest potential improvements in design methods and processes to take particularly interesting and/or important aspects of their response into account.

1.4 Outline of the Thesis

This thesis has been organised so that it follows the stages of work undertaken, detailed in chronological order to investigate the full structural response of reinforced concrete columns throughout the process of localised heating whilst under sustained eccentric axial compressive load (see Figure 1.1). Chapter 2 outlines the "state of the art" in terms of the current understanding of the response of concrete, specifically reinforced concrete columns, when exposed to elevated temperatures, with a particular emphasis on the available design guidance used. A brief outline of the procedures used is provided with a more in depth discussion of the background of these methods and limitations of each in the design and construction of modern buildings. These design methods have later been applied to the reinforced concrete columns tested herein to provide context into this investigation of the limitations of each method. Following this, the current understanding of the "real structural behaviour" of reinforced concrete columns subjected to elevated temperatures is given in terms of the current pool of data available and modelling techniques used to provide the reader with an understanding of specifically why this research project has been undertaken.

Chapter 3 outlines the experimental program conducted as part of this research project including all design decisions and experimental methods used. In summary a total of 46 tests were conducted on geometrically identical reinforced concrete columns. Various data were gathered throughout the entire experimental process, from first loading the specimen before heating, during heating, and during the cooling phase. The results of this experimental program forms the foundation of Chapter 4, and Chapter 5 where the results of the experiments in each stage of the fire described in Figure 1.1 are interrogated test-by-test in isolation.

On cooling to ambient temperature after exposure to elevated temperatures concrete structures reach a new point of equilibrium, after which it is necessary to determine the damage sustained and the remaining capacity to carry loads. Chapter 5 also looks into some of the methods available to determine the damage sustained by concrete structures due to exposure to elevated temperatures. Their accuracy in predicting the damage sustained by the columns tested is detailed in this chapter, and possible applications discussed.

Chapter 6 discusses the analytical model created by the author attempting to quantify the response of the experiments described in Chapters 3 though 5. Specifically, a sectional analysis model is discussed which has been used to determine the total strength of the columns before, during, and after exposure to elevated temperatures.

Finally, Chapter 7 provides a summary of the findings of this thesis, outlines the key conclusions, and provides recommendations for the next steps in further developing and improving the methods used in the design and construction of fire-exposed reinforced concrete columns.

Chapter 2

Literature Review

2.1 Introduction

The design of structures to resist fire is an area of research with ever increasing interest. In recent years there have been a number of high profile incidents which have brought this specific area of engineering very much into the public eye, with specific interest in preventing the spread of fire with effective compartmentation and ensuring that the structural frame is designed in such a way that it can perform adequately during a fire event. The well-established and internationally recognised method of designing structures for fire resistance around the world has, traditionally been focused on ensuring that a fire resisting element within a building will continue to perform its load-bearing and fire compartmenting duties for a prescribed time of fire resistance, as dictated by the local fire safety guidance and the circumstances in which it is implemented (i.e. building type and occupancy).

This is [2, 11–15] based upon structural and certain non-structural building elements resisting a standardised heating regime (i.e. standard fire) for the specified duration (i.e. fire resistance) before the failure criteria of the test have been met; these being load-bearing, insulation, and integrity. Only the load-bearing requirement is relevant to most columns. Therefore, a 60-minute fire resisting column will maintain its load bearing capacity for a minimum of 60 minutes of this standard fire exposure.

The "standard fire curve" used in the fire testing of structural elements dates back to the early 1900s, when it was originally developed and codified. Changes and updates to fire resistance testing equipment and procedures have been made through the years, including the development of a suite of additional standard fire curves to be applied in different situations (e.g. hydrocarbon fires, fire in tunnels, external fires, etc), but both the overarching furnace test method and compliance testing approach have remained much the same for more than a century.

This method has stood the test of time due to the fact that, in a field where the prediction of actual fire behaviour is so challenging, it has provided engineers and designers with a standardised and quantifiable way of comparing the fire resistance performance of different structural elements and typologies. It can, however, be argued that with the ever-increasing complexity of modern buildings this simplified and coarse approach to fire resistance assessment and design may provide differing levels of safety (i.e. risk) between different buildings due to the individual circumstances of each design. Engineers should perhaps then be interrogating more rational approaches to this issue in design.

This Chapter provides an overview of the current state-of-the-art regarding the analysis, design, and fire resistance testing of reinforced concrete columns, with the goal of providing context for the experimental test series discussed throughout the remainder of this thesis. The structural response of reinforced concrete columns during fire is the overarching theme of this thesis, therefore the main topics explored throughout this literature review are:

- current fire resistance design guidance for reinforced concrete and its application;
- concrete material properties at high temperature;
- concrete material properties while cooling and after cooling; and
- post-fire (i.e. residual) analysis of concrete structures (including non-destructive testing and assessment techniques).

2.2 Current Design Guidance

Currently, with regard to the design and construction of reinforced concrete elements, the Eurocode suite of design guidance is the most prominent guidance widely referenced today. BS EN 1992: Part 1-2 [2], which specifically deals with the design of concrete structures subject to elevated temperatures states that:

"*The construction works must be designed and built in such a way, that in the event of an outbreak of fire:*

- *the load bearing resistance of the construction can be assumed for a specified period of time,*
- *the generation and spread of fire and smoke within the works are limited,*
- *the spread of fire to neighboring construction works is limited,*
- *the occupants can leave the works or can be rescued by other means,*
- *the safety of rescue teams is taken into consideration".*

In most practical cases, the statement above is assumed to have been satisfied provided that the guidance within the document has been followed and applied appropriately by an engineer competent to do so. It is therefore prudent to regularly review and critique the current guidance provided for design because its very application assumes "*adequate safety*", therefore implicitly satisfying all of the statements detailed above - whether explicitly or rationally considered or not.

In the design of reinforced concrete columns, for instance, the current design guidance recommends the use of so-called advanced calculation models where required and appropriate. However, no useful detail is provided concerning what this actually entails, leaving this area significantly open to interpretation, both as regards the material models to be used (thermal and mechanical) and the structural behaviours to be considered (material, element, sub-frame, full frame, or global structural). In proceeding with an advanced calculation method for the analysis of a concrete structure or structural element, BS EN 1992-1-2 [2] requires only that:

"*Advanced calculation methods shall provide a realistic analysis of structures exposed to fire. They shall be based on fundamental physical behaviour leading to a reliable approximation of the expected behaviour of the relevant structural component under fire conditions.*"

The above raises the question, given the current state of the art, whether there is sufficient knowledge about the fundamental physical behaviour of concrete to make a reliable approximation of the expected behaviour, particularly in cases where deformations during heating may affect element and/or structural performance in a real-world scenario. This question captures the very nature of the current thesis, and is explored in greater detail in the following chapters for fire exposed concrete columns in particular - with a comparison against real structural tests conducted as part of this thesis. The experiments presented herein are of course another recommendation of BS EN 1992-1-2, where it is stated that, if undertaking an analysis using an advanced calculation model:

"A verification of the accuracy of the calculation models shall be made on the basis of relevant test results."

This raises additional questions regarding the responsibilities of practitioners in the design of concrete structures subjected to fire. To what extent should designers seek experimental evidence that validates their analysis and, given the pace at which structures are becoming more advanced and optimised, both in terms of structural design efficiencies and in terms of advancements in material properties (i.e. compressive strength), how do the structural and fire engineering communities, as industries, ensure that the knowledge available keeps pace with this advancement? Such questions are explored in greater detail in Chapter 7 as regards the design and analysis of reinforced concrete columns subjected to elevated temperatures whilst carrying sustained, eccentric compressive loads.

2.2.1 Design Methods

With respect to the aforementioned Eurocode design suite, BS EN 1992-1-2 [2] recommends one of the following methods for the design of concrete columns subject to elevated temperature. It is the responsibility of the designer to interrogate which of these is the most appropriate solution for a given design scenario:

- Detailing according to recognised design methods detailed within BS EN 1992-1-2.
    1. Method A
    2. Method B
- Simplified calculation methods for specific types of members.
    1. 500°C Isotherm Method

2. Zone Method

- Advanced calculation methods for simulating the behaviour of structural members, parts of the structure or the entire structure i.e. Finite Element Models etc.

Of the methods detailed, Method A and Method B are based on tabulated empirical data, whereas the 500^o Isotherm Method and the Zone Method allow for slightly more flexibility in design in that there are no restrictions on the dimensions of the element. They are based upon the fact that concrete loses strength at elevated temperature, therefore providing less structural resistance than the virgin, undamaged concrete. For each of these methods, the cross section is divided into zones of *damaged* and *undamaged* concrete. All subsequent structural calculations are based upon the damage map produced for the section as a result of the temperature gradient within the section (determined using tables or a conduction heat transfer analysis). Details of these methods and their validation are detailed in the following sections, discussion of the benefits and constraints of each has been provided.

2.2.2 Method A

Method A, as described in BS EN 1992-1-2 [2] is for use on reinforced or pre-stressed concrete columns subjected to compressive loads. The output of this method is a number representing the "fire resistance" of that particular concrete column in minutes. For example, if an element achieves a rating of R60, it is assumed to achieve 60 minutes fire resistance. For this method to be valid, the following conditions must be satisfied:

- effective length of the column: $l_{0,fi} < 3m$
- first order eccentricity under fire conditions: $e = M_{0Ed,fi}/N_{0ED,fi} \leqslant e_{max}$

- area of reinforcement: $A_s < 0.04A_c$

Given that the structural element being designed satisfies the three conditions stated, the "fire resistance" of the element in question may be looked up in the table provided within the guidance. Otherwise Equation 2.1 may be used for columns that fall out with the given values.

$$R = 120((R_{\eta fi} + R_a + R_I + R_b + R_n)/120^{1.8}) \tag{2.1}$$

Empirical relationships are detailed within BS EN 1992-1-2 [2] to calculate the intermediate fire resistance, R_{xx}, of elements in Equation 2.1.

Validation of Method A

In providing a critique on design guidance, there are a few considerations to be taken into account: Using what methods/data was the method developed and, given subsequent work completed in the area, determine where it is possible for improvements to be made to the design guidance. This section aims to interrogate these questions to determine exactly where this method sits in the current state of the art and determine exactly where efforts can be concentrated to increase our confidence in this approach in the design of concrete columns.

Method A in its current form was first proposed in 2000 by Franssen et al. [1] as an improvement to the methods used at the time in design. It is based upon parametric investigation of a number of reinforced concrete columns using the numerical code SAFIR [16]. This parametric study was validated using data from two test series conducted at the University of Liege [17] and the University of Ghent [18] involving a total of 82 experimental tests on reinforced concrete columns. It is clear from this comparison that the model proposed provides a consistent level of safety which is not sensitive to the value of any one variable.

Of the 82 columns tested, the fire resistance times predicted by the Method A model were conservative i.e. “safe” for 48 and “unsafe” for 34 of the tests. This means that the model is not conservative for 41% of the tests that it is validated against. The comparison of the fire resistance times calculated by this method against the actual fire resistance of the columns tested has been provided in Figure 2.1.

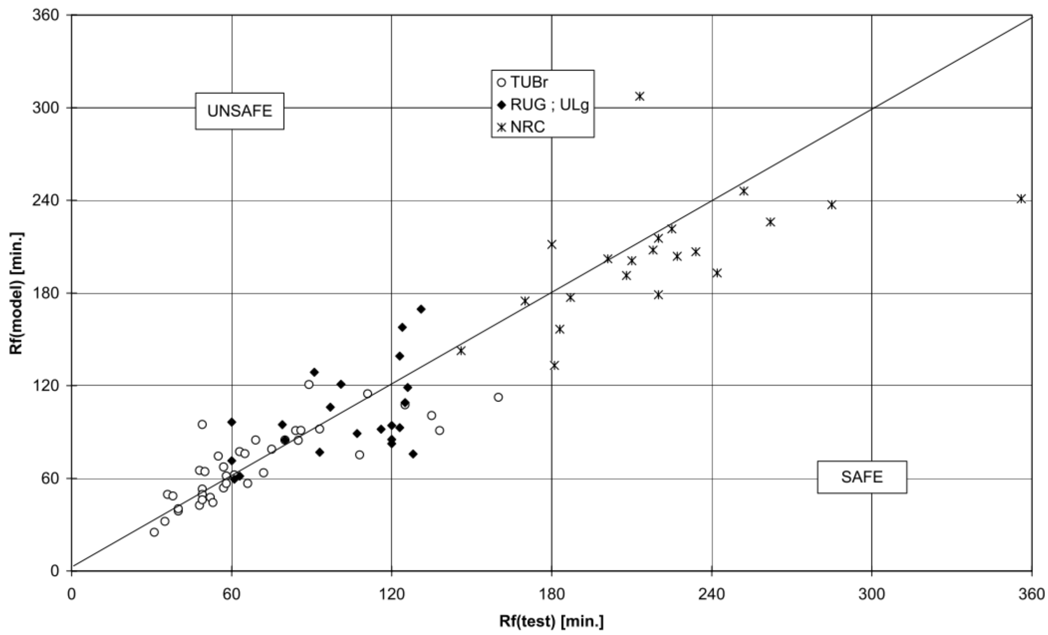


Figure 2.1: Comparison of the fire resistance (in minutes) of the method proposed by Franssen with the actual test results, reproduced from [1]

It should also be noted however that no safety factors were used for the material properties used in the model. When the appropriate safety factors for concrete and steel are applied appropriately in accordance with the guidance provided within BS EN 1992-1-2 [2], the comparison between the model and the tests becomes far more favourable.

This statement does of course come with some caveats regarding the limits of its applicability to different columns for which it has been validated and data exists:

- Load level: $0.15 \leq \upsilon_{fi} \leq 0.80$
- Dimensions of the section: $200 \leq b' \leq 450mm$ & $b_2 \leq 1.5b_1$
- Concrete cover: $25 \leq a \leq 80$
- Length of the column: $1.5 \leq L \leq 6.00m$
- Reinforcement ratio: $0.9\% \leq A_S/A_C \leq 4\%$
- Concrete strength: $24 \leq f_{cm} \leq 53MPa$
- Eccentricity: $e \leq 15cm$
- Diameter of the bars: $\phi < 25mm$

From the above list it is clear that the method has wide limits of application however, as construction methods evolve and alternative design practices become more popular (e.g. higher concrete strengths), the limits of this method are beginning to be drawn into question. Particularly its application to columns where $b_2 >> b_1$, and for high strength concrete mixes. It should also be noted that the experimental program from which the method is validated [17, 18] consist of tests conducted exposing the columns uniquely to the ISO 834 [15] fire curve around its full perimeter. Therefore, much like most of the guidance available for the design of structural elements, an assumed "worst case" fire is defined in the ISO 834 temperature-time curve, and all fire resistance times achieved are based upon the perceived performance of the element against this "worst case" scenario. This is an area of much discussion in the current fire safety climate and is one of the primary focus points of this thesis. Questions may be asked about whether it is possible to assume a "worst case" fire and design buildings based upon this assumption, or if there is the possibility of missing certain behaviour which may be to the benefit/ detriment of an element under differing fire exposures. Therefore, when designing an element using Method A, the output of this method will always be the duration that the column can survive (with respect to the fire limit state load bearing capacity) exposure to the ISO 834 fire curve, exposed over the full length of the column on all sides.

2.2.3 Method B

Method B is considered to be more conservative than Method A in its application and, similar to Method A, is based upon empirical data, providing fire resistance times based upon the minimum required dimensions of the element. It is again assumes that the fire exposure to the elements of the ISO 834 standard fire curve, which is assumed to engulf the entire column. The table provided within BS EN 1992-1-2 for has been provided in Figure 2.2.

Therefore, the output of both of the simplified methods recommended within the Eurocode design suite are that of a “fire resistance time”. These times are all based upon empirical relationships which do not explicitly take physical behaviour into account and are a representation of the performance of a given column to one very specific, severe, fire exposure when tested as an isolate element. Simplified methods such as these are required for immediate application where actual behaviour is not understood. However, it is important that this remains in the conscience of researchers and designers alike, so as to ensure that the correct steps are taken in order to improve understanding and increase the pool of test data available that can be interrogated to increase understanding the relevant behaviour. This issue will be revisited in Section 2.4 where the current data available is discussed alongside the general direction of travel of experimental investigations into reinforced concrete columns.

Standard fire resistance	Mechanical reinforcement ratio ω	Minimum dimensions (mm). Column width b_{min}/axis distance a			
		n = 0,15	n = 0,3	n = 0,5	n = 0,7
1	2	3	4	5	6
R 30	0,100	150/25*	150/25*	200/30:250/25*	300/30:350/25*
	0,500	150/25*	150/25*	150/25*	200/30:250/25*
	1,000	150/25*	150/25*	150/25*	200/30:300/25*
R 60	0,100	150/30:200/25*	200/40:300/25*	300/40:500/25*	500/25*
	0,500	150/25*	150/35:200/25*	250/35:350/25*	350/40:550/25*
	1,000	150/25*	150/30:200/25*	200/40:400/25*	300/50:600/30
R 90	0,100	200/40:250/25*	300/40:400/25*	500/50:550/25*	550/40:600/25*
	0,500	150/35:200/25*	200/45:300/25*	300/45:550/25*	500/50:600/40
	1,000	200/25*	200/40:300/25*	250/40:550/25*	500/50:600/45
R 120	0,100	250/50:350/25*	400/50:550/25*	550/25*	550/60:600/45
	0,500	200/45:300/25*	300/45:550/25*	450/50:600/25*	500/60:600/50
	1,000	200/40:250/25*	250/50:400/25*	450/45:600/30	600/60
R 180	0,100	400/50:500/25*	500/60:550/25*	550/60:600/30	(1)
	0,500	300/45:450/25*	450/50:600/25*	500/60:600/50	600/75
	1,000	300/35:400/25*	450/50:550/25*	500/60:600/45	(1)
R 240	0,100	500/60:550/25*	550/40:600/25*	600/75	(1)
	0,500	450/45:500/25*	550/55:600/25*	600/70	(1)
	1,000	400/45:500/25*	500/40:600/30	600/60	(1)
* Normally the cover required by EN 1992-1-1 will control.					
(1) Requires width greater than 600 mm. Particular assessment for buckling is required.					

Figure 2.2: Minimum column dimensions and axis distances for reinforced concrete columns with a rectangular or circular section i.e. Method B, reproduced from [2]

2.2.4 500^{o}C Isotherm Method and Zone Method

The 500^{o}C Isotherm Method and the Zone Method are recommended in Annex B of BS EN 1992-1-2 [2]. These methods are slightly more advanced than Method A or Method B, as they do not rely on tabulated data. They require two stages to the analysis. Initially a heat transfer analysis in order to gauge the temperature gradient through the section followed by a structural analysis. The sections are split into areas of "damaged" and "undamaged" areas. The 500^o Isotherm Method assumes that all concrete that has achieved temperatures of over 500^{o}C do not contribute to the structural resistance of the section, whereas all concrete below 500^o maintains its ambient properties. A similar approach is taken in the Zone method, but it is up to the designer to split the concrete into zones of equivalent temperature and strength, and conduct the analysis. This method has been taken and adapted in Chapter 6 to calculate the strength of the columns presented in this thesis.

The difficulty with these approaches are that the simplified method of calculating the structural resistance of reinforced concrete sections does not cope well with changing material properties. The 500^o Isotherm Method bypasses this by taking a binary approach to the material properties of the concrete. Otherwise, it is difficult to determine the exact failure envelope of the load-moment interaction diagram. In addition to this, it is also stated within Annex B of BS EN 1992-1-2 that:

"*This method is applicable to a standard fire exposure and any other time heat regimes, which cause similar temperature fields in the fire exposed member. Time heat regimes which do not comply with this criteria, require a separate comprehensive analysis which accounts for the relative strength of the concrete as a function of the temperature.*"

Therefore again, within the guidance available, even the approaches that are meant to provide engineers with more freedom and understanding of what may occur during a fire are only relevant for the standard fire curves. It does however allow designers to take account of some of the actual physical behaviour that would occur during a fire. It is however up for debate whether or not it allows designers to capture enough of the physical response of the section in order to make an accurate prediction. This will again be revisited when considering the response of the columns tested as part of this experimental test series described in Chapters 4 and 5.

2.2.5 Advanced Calculation Methods

With advanced calculation methods, as briefly detailed in Section 2.2, there is no great detail provided to designers with regard to what "advanced calculation models" may be appropriate for any given task. This responsibility is left within the hands of an engineer competent to draw such conclusions. This provides designers with a degree of freedom in the techniques and models that may be used in order to demonstrate an equivalent level of safety as the prescriptive guidance. However, with a lack of experimental validation, many questions can be asked of the accuracy and creditability of such models. With regard to concrete, a material characterised by its inherent variability between mixes, the most critical step in modelling the performance of an element is determining the most appropriate material properties to implement. The following sections will therefore provide an overview of the current state of the art regarding concrete material properties and generally accepted design practices to provide context for the experimental test series conducted as part of this research project.

Notice that within the design guidance, all of the methods described are provided to designers in order to determine how a concrete element will perform during a fire. The output of which is a number representing the number of minutes that it will survive being exposed to elevated temperature. Nowhere is the cooling phase of a fire taken into account. Should the structure survive for a sufficient period of time that the "fire" is put out, the response when the structure begins to cool is not explicitly considered. This is an area of design that is completely neglected but, as described in Chapter 4, it is clear from the experimental test series conducted as part of this research project that there can be equally dramatic responses of the structure during cooling as there is during the heating phase. Giving rise to questions of the safety of some buildings being entered after a fire. Therefore the post-fire performance of concrete structures is not only of interest from an insurance perspective, but also from a life safety perspective, which currently is assumed to end after the “fire resistance” period of the structure has expired.

2.3 Concrete Material Properties

Concrete is a composite material of cement (activated through the addition of water) and aggregate (and a number of additional admixtures). It can be traced back many years and the material, as it is known today, is the result of many years of research and trial and error. Many advancements in technology and structural design later, we are now capable of constructing ever growing and more complex structures using concrete. This, in turn, has greatly increased the consequences of structural failures resulting from rather rare events like earthquakes, blizzards, and fires, to name a few. Therefore, the demand for research into these areas and methods to predict, and design for such events has increased greatly over the years.

It is generally accepted that concrete performs rather well in fire [19]. Its low thermal conductivity results in a very slow thermal wave penetrating into the material resulting in a weakened layer of concrete which provides insulation for the steel reinforcement and the concrete deeper within the element. As the temperature of the concrete element differs through the depth of the column, the strength and response of the column will be a function of both the profile of the thermal profile and the properties of the concrete at different temperatures.

2.3.1 Ultimate Strength at Elevated Temperature

Often the first question regarding the performance of concrete at elevated temperature will be with regard to its ultimate strength. This is potentially the most heavily investigated material property of concrete over the years with many different mix proportions and experimental configurations being implemented [3, 20–29]. Many of the experimental test series conducted have produced correlations relating to the ultimate strength of concrete when exposed to certain temperatures. Ma et al. [3] collated a number of the test results conducted on the ultimate strength of concrete over the years into one plot. This has been detailed in Figure 2.3.

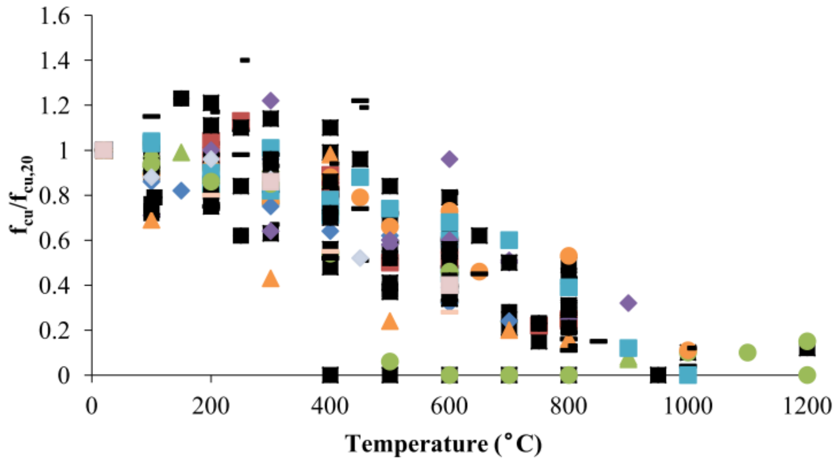


Figure 2.3: Collated test results for concrete strength at high temperature. Reproduced from Ma et al. 2015 [3]

It should be noted from 2.3 that this plot details a number of experimental test series conducted on many different concrete mixes using varying methods to expose the concrete to elevated temperature. It does however illustrate the point that, although general trends can be seen in the strength of concrete as temperature increases, the variability of concrete results in a wide scatter in Figure 2.3. This is in essence due to the fact that the word "*concrete*" does not describe a material. Rather, concrete itself is a structure of constituent materials that will interact with each other. As with any structure, altering the materials or volume of the constituent parts will result in different behaviour being observed. It may therefore be possible to, using a single concrete mix, define an accurate and reliable relationship between the strength of the concrete and the temperature achieved. However upon changing the mix proportions/ materials used, as is the case for each building/ region/ country, this relationship will be changed and is no longer entirely valid for that specific concrete mix.

In terms of design however, there is a requirement for designers to be provided with a method of predicting the reduction of strength of concrete as temperature increases. The current design guidance used to provide an estimate of the ultimate strength of concrete at elevated temperatures, BS EN 1992, Part 1-2: General rules - Structural fire design [2], provides such a correlation. Figure 2.4 illustrates the same empirical relationship proposed within BS EN 1992-1-2 to represent the reduction in strength of concrete and the stress-strain relationship at elevated temperatures

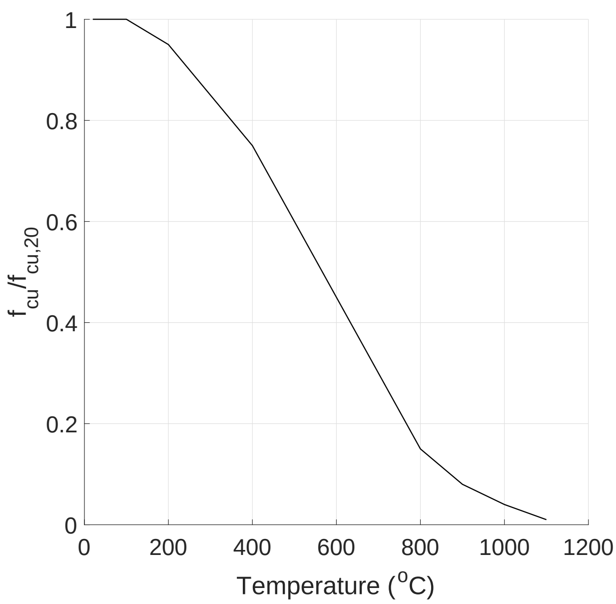


Figure 2.4: Ultimate strength of concrete at elevated temperatures according to BS EN 1992-1-2

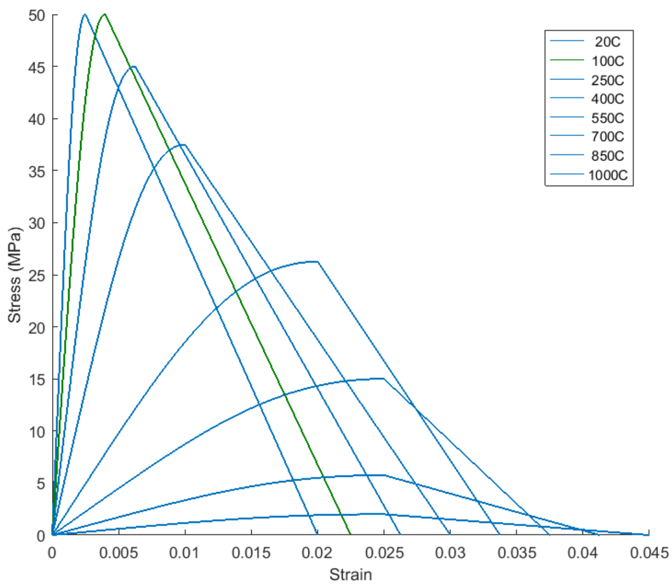


Figure 2.5: Stress-strain relationship of concrete at elevated temperatures according to BS EN 1992-1-1 [4]

The relationship between the stress and strain at different temperatures presented in Figure 2.5 is relatively well understood and can be attributed to the mineralogical changes which occur in concrete when it is exposed to high temperatures. Temperatures of beyond 105^{o}C see the loss of the physical water located in the pores of the matrix with a slight loss in strength. The loss of the physically bound water results in an interesting phenomenon in which all of the additional energy entering into the concrete is used to vaporise the water resulting in the temperature of the concrete plateauing at around 100-120^{o}C, until the water has been evaporated and the energy can again be used to increase the temperature of the concrete itself. The loss in strength becomes more pronounced at temperatures of 300^{o}C [10] as the chemical degradation of the aggregate and cement matrix intensifies, until the concrete can be considered to provide questionable value from a structural perspective beyond temperatures of 500^{o}C [10] at which point the strength of the concrete has been reduced to 50% of the ambient strength. There have been a great deal of experimental studies conducted in this area which tend to agree with the results presented in the graph above [30–33]. From these studies, it is clear that the results of the tests conducted in this area conclude that the ultimate strength of concrete decreases with exposure to elevated temperature and the corresponding strains at failure increase. This suggests that not only the strength, but the stiffness of any concrete structure subjected to elevated temperature will decrease as temperatures increase, thus increasing deflections and secondary effects on the structure in question.

The empirical relationships presented in this section are the fundamental building blocks of any analysis into the performance of concrete at elevated temperatures, and the volume of complimentary experimental work completed in this area can lend confidence to engineers that the stress-strain relationship of concrete at these temperatures is understood to a level that it can be applied to design. This statement however comes with the caveat that, to be applied in design, the other phenomena which are present in the process of heating concrete must be taken into account i.e. spalling, transient thermal creep etc.

2.3.2 Elastic Modulus

As is clear from Figure 2.5, another property of concrete that is affected by an increase in temperature is the elastic modulus of the material. This is an important parameter when considering reinforced concrete columns due to the reduction in the stiffness of the section that results from a reduced elastic modulus. This will be discussed on a number of occasions throughout this thesis as the degradation of this material property ultimately results in increased deflections and secondary moments being induced in the reinforced concrete columns tested.

A relatively large number of studies have been conducted on the reduction of the elastic modulus of concrete at elevated temperature [34–36], some dating as far back as 1966 [36]. Similar to the ultimate compressive strength discussed in Section 2.3.1, the conclusions of these studies are all very similar in terms of the behaviour however, the exact magnitude of the reduction differs rather extensively. This large range is again the result of the inherent variability of concrete resulting from the many mix designs possible. Figure 2.6 displays the results of a number of studies conducted in measuring the reduction of the elastic modulus of concrete at elevated temperature.

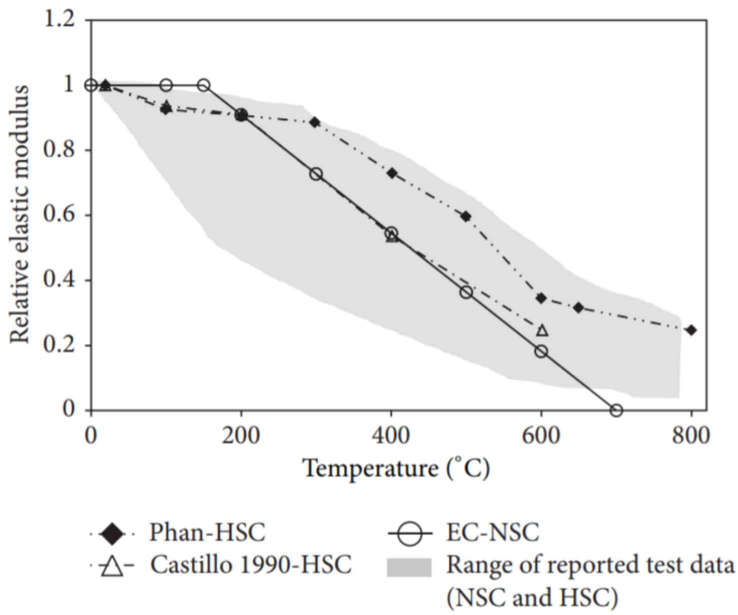


Figure 2.6: Reduction in elastic modulus of concrete with increasing temperature, reproduced from [5]

Therefore it is clear that, similar to the reduction in the ultimate strength of concrete at elevated temperature, there is definitely a reduction in the elastic modulus of concrete. The exact value of the reduction on the other hand proves to be a difficult item to definitely predict based solely on the temperature profile of the concrete.

2.3.3 Spalling

Spalling is a phenomenon observed in concrete that has been exposed to elevated temperatures and is the subject of much research today [37–39]. Spalling refers to the falling off of concrete from the surface of an element when exposed to heat due to complex phenomena involving the time and temperature dependant stresses, chemical changes in the material, moisture movement and temperature gradients [?]. Spalling is a behaviour that most definitely affects the structural performance of concrete elements, however predicting exactly when and where it will occur is a difficult task. It was first discovered in the early 1900's [?] and has been the focus of many research papers. Despite this, there remains a paucity of data available for models to be created and the prediction of spalling under different scenarios is not currently possible. As a result of the uncertainty posed by the spalling of concrete at elevated temperature, it is outwith the scope of this research project to discuss the exact process and current state of the art. Instead it is accepted that this is another facet of the design of concrete structures that has to be taken into account in addition to the resistance of the element. Indeed, it is an area of design that, if ignored, could render all of the design work irrelevant should the concrete that is relied upon for structural resistance experience spalling.

No spalling was observed during the experimental program presented in this thesis. Indeed, steps were taken, through the addition of polypropylene fibres to the concrete mixes, to minimise the propensity for spalling in the concrete mix design used. As the design of many reinforced concrete structures do not take these steps to minimise the possibility for spalling, it is a phenomenon that, despite not addressed directly as part of the research presented herein, requires consideration in the design of reinforced concrete structures, and continues to be the focus of many research programs.

2.3.4 Transient Thermal Creep

Another material property unique to concrete is that of transient thermal creep, otherwise referred to as load induced thermal strain (LITS) in some cases. Transient thermal creep, despite being a well recognised phenomenon, is not well understood and the exact application of these material models into a structural system is yet to be extensively validated with in depth experimental studies. This area is one which presents itself later in this thesis during the reinforced concrete column experimental test series detailed in Chapter 4 due to the fact that a relatively large compressive stress is induced in the concrete column during the process of heating, a key condition for the onset of load induced thermal strain.

Description of Transient Thermal Creep

Transient thermal creep is a constituent strain that can be observed when concrete is stressed to a particular level and then exposed to elevated temperatures. This results in non-reversible strains which remain present after cooling [6].Take a standard concrete cylinder for example, this concrete cylinder has been illustrated in Figure 2.7. The cylinder is at ambient temperature, when a load is applied to the cylinder, the stress results in deformation and the corresponding strain, $\Delta\varepsilon_{ela,0}$, can be calculated from the stress-strain relationship of the concrete itself. Provided that the concrete has not transitioned far into the plastic range, when the load is release, the mechanical strain will return to zero. As illustrated in Figure 2.7, when no load has been applied and the temperature is increased, it can be seen from the literature that thermal expansion occurs as a result. Thermal expansion of concrete is well investigated and a number of correlations have been proposed to predict the thermal strain, ε_0, induced by the increase in temperature. As with the mechanical strain, as the concrete cools back to ambient temperature, the thermal strain is recovered and returns to zero upon getting back to 20^{o}C.

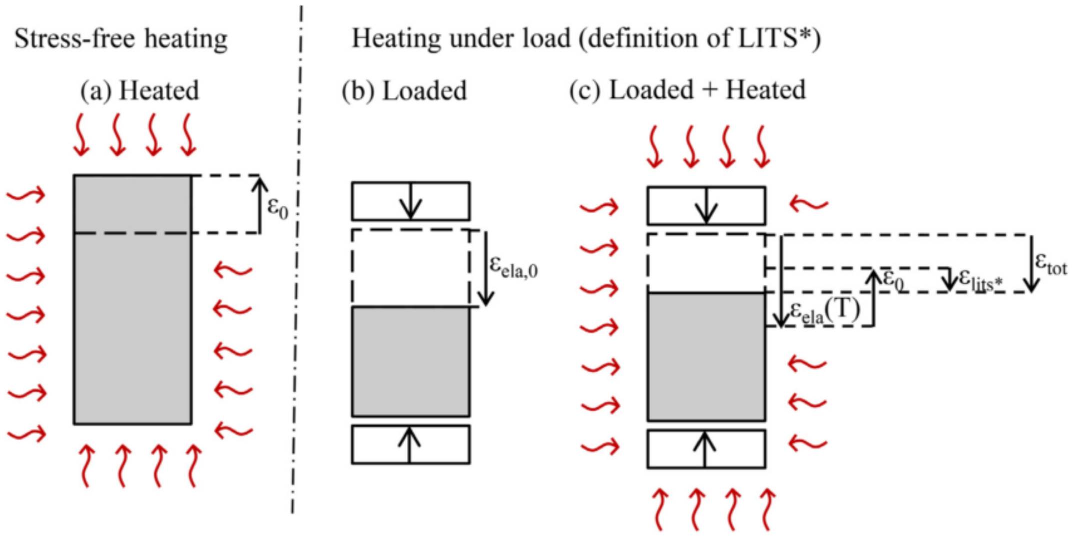


Figure 2.7: Illustration of LITS. Reproduced from [6]

However, should the cylinder be loaded to the equivalent stress state and heated at the same rate, to the same temperature, the simple addition of the mechanical and thermal strains will not predict the real strains achieved in the concrete. After being loaded, the mechanical strain in the concrete can be observed and upon heating, thermal expansion will begin to occur until temperatures of around 105^{o}C. At this point it can be seen that the concrete will begin to "shrink" i.e. as opposed to the expanding, as predicted by the thermal expansion material model for concrete, the strains in the concrete begin to compress. There is therefore an additional mechanism taking place within the concrete that results in transient thermal creep, or LITS, ε_{lits}.

Transient thermal creep has been researched for around four decades now Papers dating back to the early 1980's describe the importance of both understanding, and confidently predicting this behaviour in concrete structures likely to be exposed to elevated temperatures [40–43]. It is however clear that engineers are yet to develop material models robust enough to be confidently applied to structural models involving concrete elements exposed to elevated temperatures.

A thorough state of the art review of concrete strains under transient thermal conditions was conducted by Torelli et al. [44] from which it is clear that the interest in the strains experienced by concrete at elevated temperature has been gaining popularity in recent years with a number of predictive models being developed for predicting this behaviour along with comparisons to concrete cylinders [6, 45–49]. This has lead to a number of different transient thermal creep models developed and used modelling campaigns to validate for more complex scenarios. Gernay [50] completed a modelling campaign to validate an explicit thermal creep formulation for the BS EN 1992-1-2 model using the experimental data of Wu et al. with some success. This is a step in the direction of explicitly considering transient thermal creep in design as opposed to continuing to use assumption that BS EN 1992-1-2 takes account of this "implicitly" as a result of being validated against tests where it would have occurred.

Transient thermal creep is of particular interest in the research presented herein due to the fact that compressive elements are tested at elevated temperatures. These conditions are theoretically ideal to induce transient thermal creep in the experiments. This therefore provides data from which the material models developed can be validated against to determine whether this is an important property to be included into Finite Element models to predict structural behaviour.

2.4 Experimental Tests on Concrete Columns

As it stands there have been a number of experimental test series on concrete columns, and the performance of concrete at elevated temperature in general. This has resulted in a great number of correlations relating different material properties of concrete at certain temperatures, as previously discussed. This however is not the case for the performance of concrete columns at elevated temperature. For the volume of work conducted on concrete in general, there is a deficit of experimental results of concrete columns that can both; be used for the validation of models produced and, use credible, modern techniques to collect data and determine performance. Much of the current work that has been completed could be seen as rather disjointed with regard to the parameters being tested and testing methods themselves. Many of the current experiments conducted in the literature will investigate the effect of a specific parameter with a single test frame/setup and draw conclusions which may not be applicable for a different test setup or column design.

One example of this would be Lie and Lin [51] who investigated the influence of restraint on the performance of reinforced concrete columns at elevated temperature. "Performance" in this case is defined as the time to failure of the column in which case the conclusion was that the restraint conditions do not have a significant effect on the time to failure of the columns. This conclusion, although true for this specific test setup, may not have been reached for a different heating regime with a different temperature-time history throughout the column or a different load eccentricity which would, in turn, influence the increase in load due to thermal expansion as a result of the differing stress through the section. This is of course a historical test conducted in 1986; however it does somewhat set the tone for much of the experimental work conducted since.

Much of the research conducted since has concentrated on testing reinforced concrete columns in one arrangement or another within a furnace while being exposed to the ISO 834 fire curve. These works have, without exception, quoted the failure of the elements in terms of minutes from the onset of fire. Dotreppe et al. [17] conducted a large test series at the University of Liege and the University of Ghent. This test series is the series used to validate Method A for the design of concrete columns presented in BS EN 1992. It was an innovative test series in the fact that its primary aim was to provide evidence that parameters other than the dimensions of the cross section and the values of concrete cover affected the performance of concrete columns in fire. This papers states a number of conclusions on the performance of the concrete columns tested, all of which are however related to the time to failure, in minutes. A criterion that has no actual physical meaning. Referencing the fire resistance in terms of minutes, despite its practical application in construction, is not an accurate, scientific definition of a structural element that can be used in research. As a result of the lack of detailed temperature data from thermocouples within the concrete, and displacements of the element, the test data is difficult to interpret when considering the physical interaction of the columns and the thermal or mechanical loads applied. Therefore only empirical relationships may be determined to relate the "fire resistance" of the columns back to the time (in minutes) that it survived the ISO 834 fire curve.

Other experiments conducted on the performance of reinforced concrete columns have considered the occurrence of spalling on the "fire resistance" of the element. Spalling is an area of concrete research which will not be discussed as part of this literature review, with the exception of the experiments conducted on concrete columns [18, 52, 53]. These test series have, unlike others, collected some data on the internal temperatures of the section and the displacements, however the final conclusions are again related to the "fire resistance time".

The study conducted by Kodur et al. (2003) [54] marks somewhat of a turning point in experiments conducted to gain an insight into the performance of concrete columns at elevated temperature [54]. Within this study, a series of six full size reinforced concrete columns were subjected to a mechanical load, before being subjected to the standard fire curve 45 minutes later. The heat exposure was maintained until failure of the columns, or until deflections could no longer be measured. This approach, despite still relying on the standard fire curve to quote the "fire resistance" of each of the columns, has collected much more detailed data on the internal temperatures of the sections and the deflections of the columns.

Other than the conclusions made at the time, despite relying on fire resistance times, this data can ultimately be used in the modelling campaigns undertaken in the future. The main caveats with this paper however are that only six specimens were tested, offering a somewhat limited pool of data to be compared, but valid, useful data nonetheless.

This trend of slightly more detailed data collection is however not standard. More recent studies continue to neglect the importance of the internal conditions of the concrete when testing for "fire resistance". Benmarce and Guenfoud [55], and Bikhiet et al. [56] again conduct experimental tests on reinforced concrete columns by measuring the external conditions of the elements with no data collection of the internal temperatures. Therefore the conclusions taken from such papers are useful in that they provide additional qualitative data for the performance of concrete columns during fire, but quantitative conclusions that can be taken forward by designers are difficult to make from such tests. Other than this, test series have been on columns under differing arrangements and cross sectional areas [57, 58]. It could be argued that, with the inadequate volume of data available, that priority should be placed on understanding and predicting the behaviour of simple structural arrangements before moving to more complex scenarios where more complex geometric considerations have to be taken into account. This will allow more in-depth conclusions to be drawn from such tests, for example when the elements do not survive as long as those with a more simple geometry, it will be possible to determine why. This is opposed to concluding simply that the more complex shapes are less fire resistant than the simple square sections.

Most recently, further experimental test series conducted by Kodur et al. (2017) [59] investigate the residual performance of reinforced concrete columns exposed to specified design fires, with both displacements and internal temperature data gathered. In terms of the data gathered, this test series, despite consisting of only two elements, provides the community with much more credible data to be used in the validation of models purely due to the fact that both the external and internal condition of the columns have been measured. It does however, like all other tests conducted on similar concrete elements, subject the elements to elevated temperatures via the use of a furnace. Provided that the internal temperature of the furnace was tightly controlled, this should make no difference when conducting a possible heat transfer analysis. However, if the internal conditions are non-uniform, this may result in the column being exposed to conditions other than what is expected from the design fire imposed.

Therefore, looking at the previous experimental campaigns conducted in the determination of reinforced concrete columns at elevated temperature, there are clear trends that can be observed from the experiments conducted. Previous campaigns provide definite benefit to the community with regard to the data collected however, there is one aspect of fire resisting testing which, in the author's opinion, seems to hinder the advance of knowledge in this area. Quoting the "fire resistance" of an element in terms of the number of minutes it has survived the ISO 834 fire curve is currently the gold standard in construction. In order for companies to sell "fire resisting" products and structural assemblies, they must satisfy certain test criteria which quotes the "fire resistance" of the product in terms of minutes surviving the ISO 834 fire curve. This however does not mean that it should be taken as the gold standard in research, quite the opposite actually. It is a standardised approach developed in the early 20^{th} century as a way of comparing "like for like" for different systems and assemblies. It is not a research tool that, after interrogation, will help researchers to understand the physics behind certain behaviour. In order to achieve this, far more data needs to be collected during the experiments. In order to interrogate a structural experiment the applied mechanical load and the structural response need to be measured. The same is true for a fire resistance test with the addition of the thermal load applied and the thermal response of the element. Thorough, meticulous data collection in this area is the only way to advance knowledge and confidently compare the models produced to the experimental results. Research should not be concentrating on comparing times to failure and other non-physical criteria which, despite holding a valuable place in construction and application, does not belong in research campaigns where the intent is to understand physical behaviour.

2.5 Models Predicting Behaviour of Concrete Columns

Given the limited volume of data available from tests conducted with the intention of determining the behaviour of reinforced concrete columns in fire, there is limited opportunity for validation of models created for the purpose of prediction of this behaviour. However, there have been a number of numerical studies conducted to this end using the data currently available. Much of these comparisons have been made on the historical tests conducted by Lie and Lin in 1985 [51] due to the fact that, as discussed in the previous section, many of the experiments conducted since have collected a sparsity of data to be used in such comparisons.

Some of the earliest models developed for the design of reinforced concrete columns [17, 53] resulted in the derivation of simplified relationships between the dimensions and material properties of the columns and their resulting fire resistance. The tabulated models developed are, to this day, used in the design and construction of concrete columns and are in themselves validated against a large number of experimental tests. These models do not however predict the physical behaviour of the columns and are instead tools for predicting the columns "fire resistance" to the ISO 834 fire curve or similar.

In the absence of experimental data to be used for comparisons, a number of researchers have developed simple models, and compared the simplified method to other methods developed or to computer models [60–63]. This allows engineers to determine, of all the methods developed, which is the most/ least conservative, but analysing each of them in a more critical manner is a difficult task.

After the tabulated data, slightly more advanced analytical models were produced in order to determine the interaction diagrams for reinforced concrete columns. Law and Gillie [61], Bajc et al. [64], Bamonte and Monte [65], and El-Fitiany and Youssef [63] each produced independent methods for predicting the load-moment interaction of reinforced concrete sections subjected to elevated temperatures. Each model specifically details methods of overcoming the difficulty of determining exactly where the failure envelope of the interaction diagram lies. Each method provides engineers with different methods to simplify the problem and overcome certain difficulties however, as with the models described previously, all comparisons are made with other methods or models that have been previously developed as opposed to experimental data. This however, as described in Section 2.4, is a difficult problem to solve given the current sparsity of data. A similar approach in determining the load-moment interaction of a column subjected to elevated temperatures has been developed as part of this thesis. This has been described in Chapter 6.

Finally, in recent years, the most “advanced” models predicting the behaviour of reinforced concrete column have been developed, predominantly using different FEM software [65–68]. These models have shown some degree of promise in predicting the behaviour of concrete elements exposed to fire. Each of the models implements different constituent material models considering; degradation of the concrete, thermal effects and, in some cases, have begun to consider transient thermal creep. There is however, still a tendency for researchers to lump the performance of certain elements down to the “time to failure”. This is an area of fire resistance testing that may, on occasions, lead to ambiguous conclusions such as; increasing load eccentricity reduces the fire resistance of an element by 20%. Although this statement may be true under one specific scenario, as is clear from the results of the experimental test series conducted as part of this research project, described in Chapters 4 and 5, there can be large differences in the performance of an element that are both to its detriment or benefit depending on the entire structural system, not just one parameter that has been altered. Therefore care needs to be taken when comparing the “fire resistance time” of different structural elements analysed.

2.6 Post-Fire Analysis

Thus far, this review has concentrated on the performance of concrete/ reinforced concrete structures while being exposed to elevated temperatures i.e. during the course of a fire. As concrete structures tend to perform rather well in this respect, an important aspect of the analysis into the “performance of concrete in fire” is the response during cooling, and the resulting residual behaviour of the structure.

There are a great number of techniques available to determine the “damage” sustained by a concrete structure during a fire. It would be normal to employ a number of these methods to determine the degree and location of damage to a structure as a result of fire. Generally, these assessments fall into one of two categories [10]:

1. non-intrusive techniques to estimate the severity of the fire or material properties of the concrete (non-destructive), or
2. taking samples and testing the damaged concrete directly to determine the residual material properties (partially destructive).

Discussing all of the testing techniques currently available is beyond the scope of this thesis. With regard to this literature review and the experimental test series conducted, the Ultrasonic Pulse Velocity (UPV) method will be discussed in more detail.

2.6.1 Ultrasonic Pulse Velocity

Ultrasonic Pulse Velocity testing is a non-destructive test that can be conducted on site in order to determine the material properties of in-situ concrete [69]. The velocity of an ultrasonic wave within concrete is directly affected by the strength and stiffness of the concrete. Therefore, for each concrete mix, an impractical relationship can be determined between the pulse velocity of the wave and the strength of the material [70]. Guidance on obtaining pulse velocity measurement from a structure and interpreting them to determine empirical relationships detailed in BS EN 12504-4:2004 [71]. One which is not discussed within the guidance however, and for which there is a limited volume of data available, is the determination of the strength of concrete after exposure to elevated temperatures using this method.

The first tests conducted to determine if this method could be used to assess fire damaged concrete date back to the early 1990's [72]. Nassif et al. [72] measured the pulse velocity of a series of concrete cores after exposure to elevated temperatures and noted that the pulse velocity of the wave was far slower in specimens that were exposed to elevated temperatures. The authors did not move any further in trying to correlate the pulse velocity with the strength of the concrete or the temperature achieved within the specimen.

This work was taken slightly further by Cioni et al. [73] with a case study on the fires in two Tuscan paper mills where a general procedure was proposed for the post-fire assessment of concrete structures. This involved the use of UPV to determine the most heavily damaged regions of the structure by finding the areas where the pulse velocity was at its slowest. The researchers could then use this information to concentrate their efforts in this area. Therefore, it is clear from this paper that the method can clearly be used for qualitative purposes, but quantitative methods of determining damage from the method were not investigated further.

More recently UPV has been investigated by a number of researchers in order to determine the correlation between the temperature achieved within the concrete and the pulse velocity of the wave [74–78]. Each of these investigations has been conducted on concrete cores heated to a uniform temperature and allowed to cool back to ambient, at which point the pulse velocity within the concrete is measured. The overwhelming conclusion of these papers is that, as the maximum temperature achieved in the concrete increases, the speed of the pulse velocity decreases. The correlations achieved between the papers were however different due to the different concrete mixes used. Therefore it is clear that a "one size fits all" solution for the correlation between pulse velocity and concrete strength is not possible. It must be investigated on a case by case basis due to the fact that the change in the material properties of the concrete, inherently changes the pulse velocity of the concrete.

Trtnik et al. [79] made an interesting attempt to overcome the empirical nature of UPV in their 2009 paper, which presents the use of artificial neural networks to predict the strength of concrete based upon its mix proportions, external parameters and pulse velocity measurements. These tests were however not conducted with the intention of applying the model at elevated temperatures. It resulted in fairly accurate predictions of the strength of the concrete and it is conceivable that, given an appropriate volume of training data, this may potentially be applied for concrete after exposure to elevated temperature.

It should however be noted that this approach to the prediction of concrete behaviour, one which seems to be gaining popularity in recent years [80–87], should be taken forward on a cautionary basis. Artificial Neural Networks are, by definition, black boxes. In producing a neural network to predict the behaviour of concrete, it will approximate a function based on a number of input parameters and training data however, in studying the weights assigned to each of the input parameters it is not possible to determine the physical meaning and implications on the actual behaviour of an element. As a result, applying this type of method on real buildings without an underlying awareness of the parameters and their physical behaviour under certain circumstances is a difficult, and possibly dangerous endeavour.

A recent case study in the assessment of a fire damaged exhibition hall used UPV in an attempt to map the damage caused by fire over the length of a number of concrete beams and columns [88]. This again allowed the authors to gauge the severity of the damage in a number of locations but the next step to use this data in to actually predict the strength of the concrete was not taken.

As most of the current volume of work conducted on the application of the UPV method has either concentrated on the assessment of concrete cylinders or cores taken form a structure, there is the potential for this method to be tested to assess whether or not it can be used to obtain more in-depth data on an actual structural element that has been exposed to elevated temperatures. To date, the data measured have been used on a qualitative basis only. Therefore, as part of this research project, the ultrasonic pulse velocity of the reinforced concrete columns tested has been measured residually to assess the maximum temperature achieved at each location in the columns and compared to the actual measured data logged by thermocouples to determine its accuracy. A detailed description of this process and the results have been detailed in Chapter 5.

2.7 Discussion of Current State of the Art

This Chapter has provided an overview of the current literature available relevant to the performance of reinforced concrete columns subjected to elevated temperatures. The overarching theme of the literature reviewed as part of this work has been that there have been a number of models created for the prediction of the response of concrete structures to elevated temperatures. These models have however been validated against a limited data bank or compared against other models which predict similar behaviour. This illustrates the current need for additional test series with methodical data collection to build a larger data bank from which modellers can benefit.

In reviewing the literature available on the performance of reinforced concrete columns during and after a fire, it is clear that the lack of data is in part due to a lack of experimental test series being conducted on this type of element. However, many test series that have been conducted have collected a sparsity of data due to difficulties in doing so for the type of tests conducted. Fire Engineering as a discipline has very well-defined approaches for determining the "fire resistance" of different elements and materials. In Structural Fire Engineering these are known as the "standard fire curves" and "fire resistance ratings", an essential suit of temperature time curves that allow practitioners to conduct standardised tests to compare "like-for-like". This does not however imply that the standard fire curves are suitable in determining the response of a material/element/structure to fire in every credible worst-case fire scenario. In the author's opinion, this is a mentality that significantly hinders innovation within Fire Engineering.

In order for academics and researchers to define credible design techniques to be used in specifying structural elements, research into the performance of elements during, and after fire must concentrate on the physics of the problem. Many of the current specifying test series simply using the ISO 834 fire curve to investigate how long the element lasts. This alone will not advance the state of the art. That is not to say that the use of the ISO 834 fire curve is invalid. Instead, the purpose of using a specific fire curve should be very carefully defined. In many structural applications, it could be said that the specific fire curve used does not matter as long as the data is methodically collected. As it is currently not possible to accurately impose what would be an accurate representation of a "fire" on a structure, it could be argued that it is not worth trying. Instead, the very purpose of research is to understand the physics of the processes that take place. The most important aspect of the experiment is therefore the data gathered. For concrete columns this relates specifically to the temperature gradient within the section, and the deflections. This data collection will allow researchers, no matter what fire curve or heating regime is used, to reliably test and develop models and algorithms to predict actual physical behaviour. This is opposed to developing empirical relationships which are useful in construction for the time being, but describe non-physical, out of date processes which are ultimately preventing the field of structural fire engineering from moving beyond the turn of the 20^{th} century.

Chapter 3

Experimental Test Setup

3.1 Introduction

The current state of the art for the design and analysis of concrete in general, and more specifically for reinforced concrete columns, has been discussed in Chapter 2. Chapter 2 shows that knowledge gaps exist that are yet to be filled with regard to the effect of different parameters contributing to the total damage inflicted on a loaded concrete element during heating and subsequent cooling. This section describes the experimental phase of the research project presented in which a total of 46 geometrically identical reinforced concrete columns were tested under a variety of loading and heating conditions to shed light on some of the factors outlined within Chapter 2. The purpose of the test series conducted was to:

- determine the effect of a range of parameters on the total damage sustained by the concrete elements during severe (localised) heating (see below for specific parameters);

- determine the effect of these parameters during the cooling phase after columns were subjected to severe heating;
- interrogate the validity of specific design methods and calculations used in the analysis of reinforced concrete columns for structural fire response and post-fire structural analysis, and;
- test certain in-situ, non-destructive techniques used to quantify damage caused by a fire.

To begin the investigation into the items detailed above, an extensive experimental tests series has been designed and undertaken to study element response, to increase the pool of data and begin to gather meaningful conclusions that can be used in the design of future experimental test series, structural fire design and assessment guidance documents. This Chapter aims to describe the methods used and reasons behind the approach taken in the current investigation. The results of these experiments have been detailed separately in Chapter 4 and Chapter 5.

3.1.1 General Description of Experimental Program

To meet the objectives outlined in Chapter 1 of this thesis, a total of 46 geometrically identical reinforced concrete columns were constructed to investigate the influence of different loading/heating conditions on the response during fire, and the post-fire structural response and performance of the columns. This also allowed for post-fire non-destructive damage assessments to be made to gather data on the accuracy/suitability of some analysis techniques used to predict fire damage to concrete. Figure 3.1 illustrates the dimensions of the reinforced concrete columns constructed as well as a general schematic setup of the bespoke testing frame that was designed specifically for this test series (additional details of dimensions and instrumentation in subsequent sections).

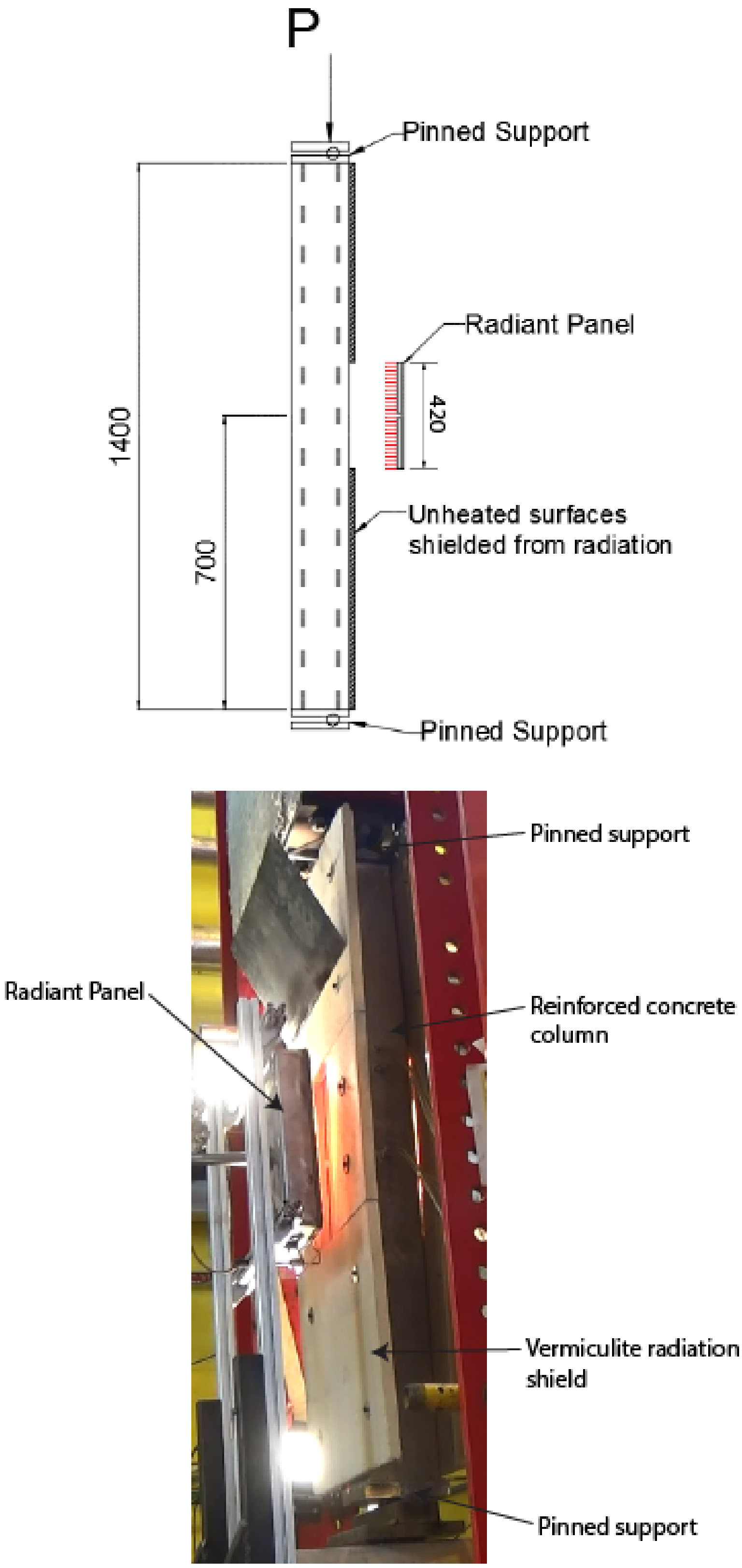


Figure 3.1: Schematic and image of the test setup

The experiments were performed in five distinct steps:

Step 1: Apply a specified load to the column at a pre-defined initial imposed eccentricity, and then hold this load constant throughout the experiment (in a load-controlled mode).

Step 2: Expose the central region of the column to a constant incident radiant heat flux, using propane fired radiant panels for a total duration of 90 minutes (provided it did not fail during heating).

Step 3: Remove the imposed heat flux and allow the column to cool under ambient conditions within the laboratory for precisely 24 hours.

Step 4: Provided that the column has not failed during cooling, undertake an assessment of potential fire damage to concrete using non-destructive techniques to attempt to quantify the degree of damage to the column.

Step 5: Destructively test the column 24 hours after heating to determine the post-fire structural response and overall performance.

Following these steps, using the data gathered, it was then possible to quantify the effects of the various parameters being investigated during fire, and post-fire response and performance of the columns. The following sections will introduce the specific experimental techniques used and define the specific parameters and load cases investigated in additional detail.

3.2 Column Design and Data Acquisition

The reinforced concrete columns used in the current study were designed in accordance with Eurocode 2 (BS EN 1992-1-1) [4]. They were roughly 1/3 scale and were symmetrically reinforced with deformed steel reinforcing bars at their corners. Figure 3.2 gives section and elevation schematics of the columns. Forty-eight geometrically identical reinforced concrete columns were cast in two sets of 24. However, as a result of honeycombing during the casting process, only 46 experiments were conducted. All of the columns were 150mm x 150mm in cross-section and 1400mm long, longitudinally reinforced with four 10mm diameter deformed reinforcing bars and 6mm diameter deformed steel ties spaced at 140mm on centre. One set of concrete columns is lower strength concrete mix (24MPa measured cylinder strength at 28 days) and second set was cast using higher strength concrete (40MPa measured cylinder strength at 28 days). It is noteworthy that both mixes included 2kg/m^3 of polypropylene (PP) fibres with the intention of minimizing the propensity for heat-induced explosive concrete cover spalling during the tests (which turned out to be successful, as noted in Chapter 4).

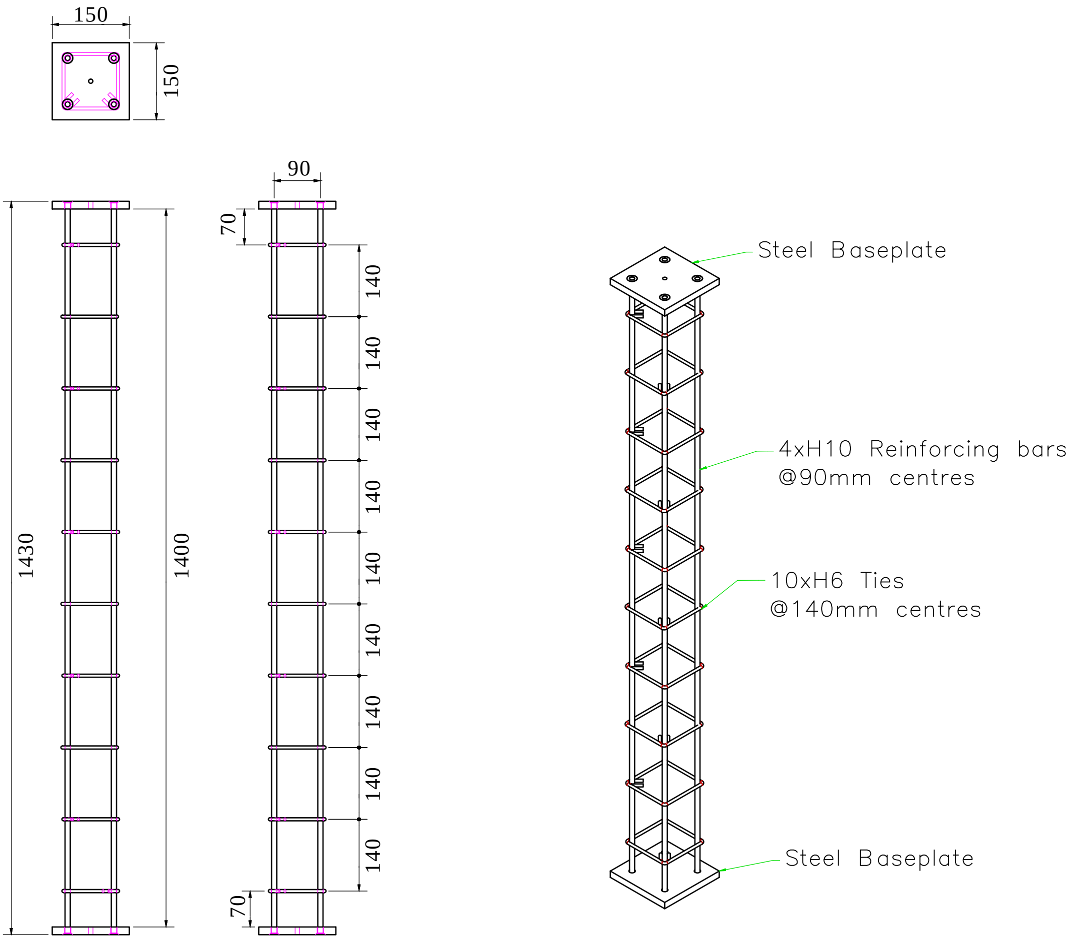


Figure 3.2: Schematic of reinforcement cage for the concrete columns cast

3.2.1 Concrete Mix

Two separate concrete mixes were used in the design and construction of the columns. The target nominal compressive cylinder strengths for the concrete columns were 40MPa for 24 of the columns and 80MPa for the remaining 24 columns. This was intended to allow the effect of different concrete strengths to be investigated as part of this series of experiments (because this affects the proportion of total element loading capacity attributed to the concrete versus the steel bars). An external ready-mix contractor was commissioned to design and deliver the concrete used to fabricate the columns. The final concrete delivered was well below the target strength for both mixes. At 28 days, cylinders cast from the two batches of concrete achieved a strength of 24MPa and 40MPa for the weaker and stronger concrete mixes respectively. Five 100mm cube specimens were also tested for each concrete mix indicating strengths of 30MPa and 50MPa respectively (stronger than the cylinders, as is to be expected when testing cubes). The standard deviation in the strength of the concrete was 0.54MPa for the weaker concrete samples and 3.13MPa for the stronger concrete samples. The remainder of this thesis will refer to the two concrete mixes used as the 30MPa mix and the 50MPa mix respectively.

As discussed further in Chapter 4, as the mix design of the two concrete types was different, it is not only the axial compressive strength of the concrete that was varied. Many other relevant properties of the two concretes were different as a result of the different mix designs; however, this allows the effect of the other parameters investigated to be tested between different concrete mixes so as to gauge general trends and help inform future test series and structural fire design of concrete elements.

Casting and Curing

All of the concrete columns were cast vertically, in two batches of 24. Each formwork assembly used held three columns and was constructed in such a manner that three sides were sealed and one side was left open to allow access for vibrating the concrete. During the process of casting the concrete columns, the formwork was filled to one third of the height of the column and vibrated to achieve good consolidation. The formwork was then sealed to a level of two thirds of the height of the column and concrete was filled another third of the column height before being vibrated once again. The top third of the column formwork was then sealed and the concrete was filled to the top and vibrated one final time before the reinforcing bars were secured and the set of three samples was left to cure. See Figure 3.3 for details of a typical formwork used to cast the concrete columns.

Figure 3.3: Column formwork used in casting

The first lower concrete strength columns (30MPa mix) were cast on 11th December 2015, and the samples were cured within The University of Edinburgh's Structures Laboratory for 10 days (formwork removed after 5 days) at a temperature of 18-22^{o}C and a relative humidity of 45-55% before being transferred to a temperature and humidity controlled conditioning room at 20^{o}C and 50% relative humidity. The concrete columns remained in the conditioning room for a minimum of twelve months until the date they were tested.

The same process as above was followed for the second batch of higher strength concrete columns (50MPa mix). These were cast on 15th January 2016 and stored in the same conditions as described above.

Securing Steel Reinforcement

The manufacture of the steel plates used on the ends of the columns also enabled the steel reinforcement to be held securely during the casting process. The steel reinforcement was welded to the plate at the base of the column. This held the reinforcement securely in the location required at the base of the column during the casting process. Immediately after the columns were cast, the other steel plate was secured to the top of the column to hold the reinforcement in the correct location during the curing process. There was therefore a large degree of confidence in the location of the reinforcement within the concrete because they were held securely in place by the machined steel plates at each end.

3.2.2 Load Application

End Plates

To ensure an even load transfer from the loading frame to the concrete columns and avoid stress concentrations due to load introduction, steel end plates were welded to the steel reinforcement bars at the tops and the bottoms of the columns as detailed in Figure 3.2. The plates were cast in-situ at the base of the columns (i.e. inserted into precision-machined holes and welded to the column base plates before concrete casting). After casting, the columns were capped with high strength plaster to ensure a smooth, snug finish, and the top plates were then secured; with the reinforcing bars inserted into the machined holes in the column base plates and welded in place. This process of securing the steel end plates to the top and the bottom of the columns was critical in that it ensured:

- the longitudinal reinforcing bars were located in precisely the correct locations within the columns, thus avoiding experimental uncertainty as regards both heating of the reinforcing bars and axial-flexural effects associated with inadvertent eccentricities of the longitudinal bars;
- localised failure at the end of the columns did not occur due to high point loads, and;
- the load was transferred over the entire surface of the column.

The design of the steel end plates placed at the top and bottom of each of the columns is shown in Figure 3.4.

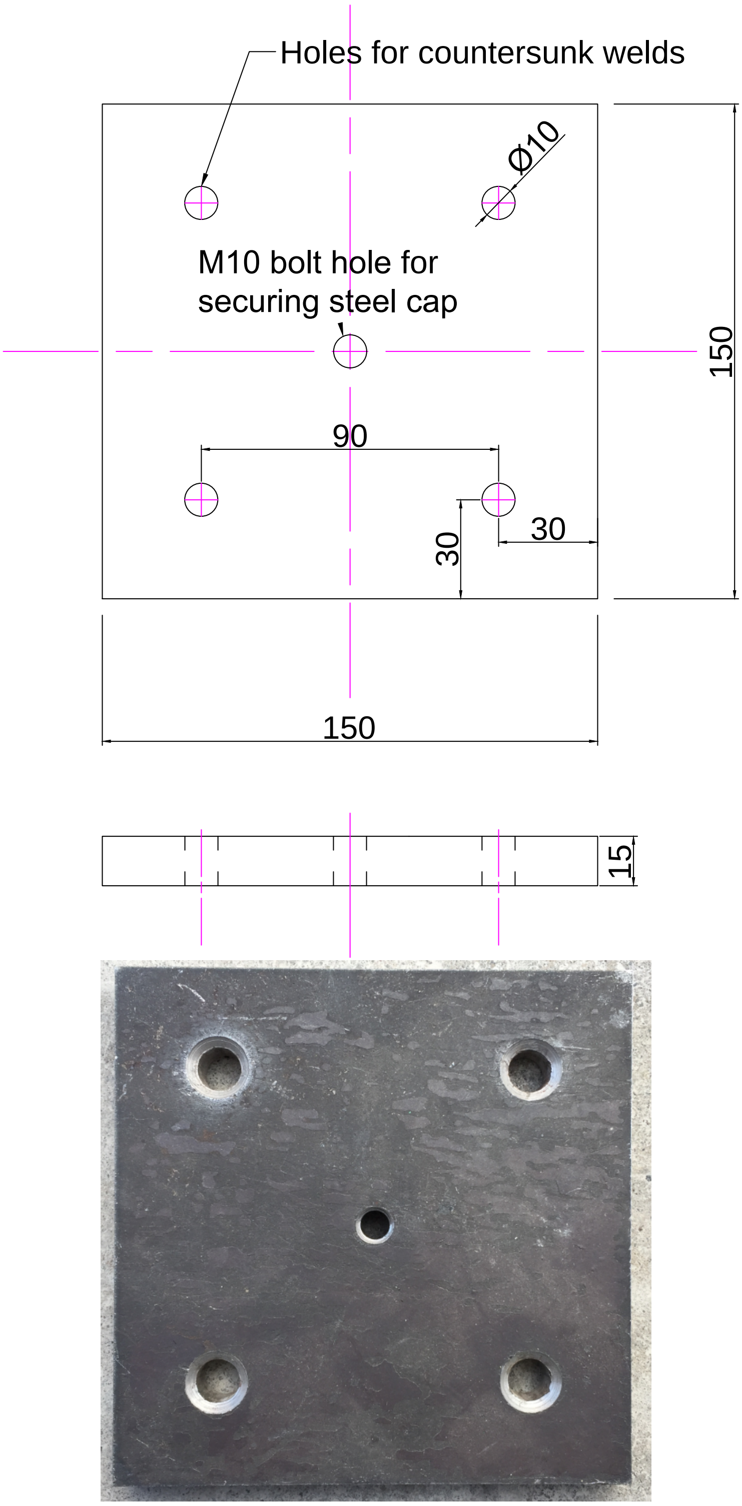


Figure 3.4: Steel plates for the top and bottom of the columns

Capping

In addition to the steel end plates secured to the top and the bottom of the columns, steel caps were manufactured to provide a consistent, precise, and repeatable method of applying sustained eccentric axial compressive load to the columns during heating and cooling. Unlike the end plates, that were welded to the steel reinforcement, the caps were designed in such a way that they could be slipped over the top of the end plates and secured with a single M10 positioning bolt. This allowed these caps to be reused for each of the experiments, thus ensuring a consistent load transfer mechanism throughout all of the experiments, along with good control of the application of the same load eccentricities across all of the tests. See Figure 3.5 for details of the steel caps manufactured and used.

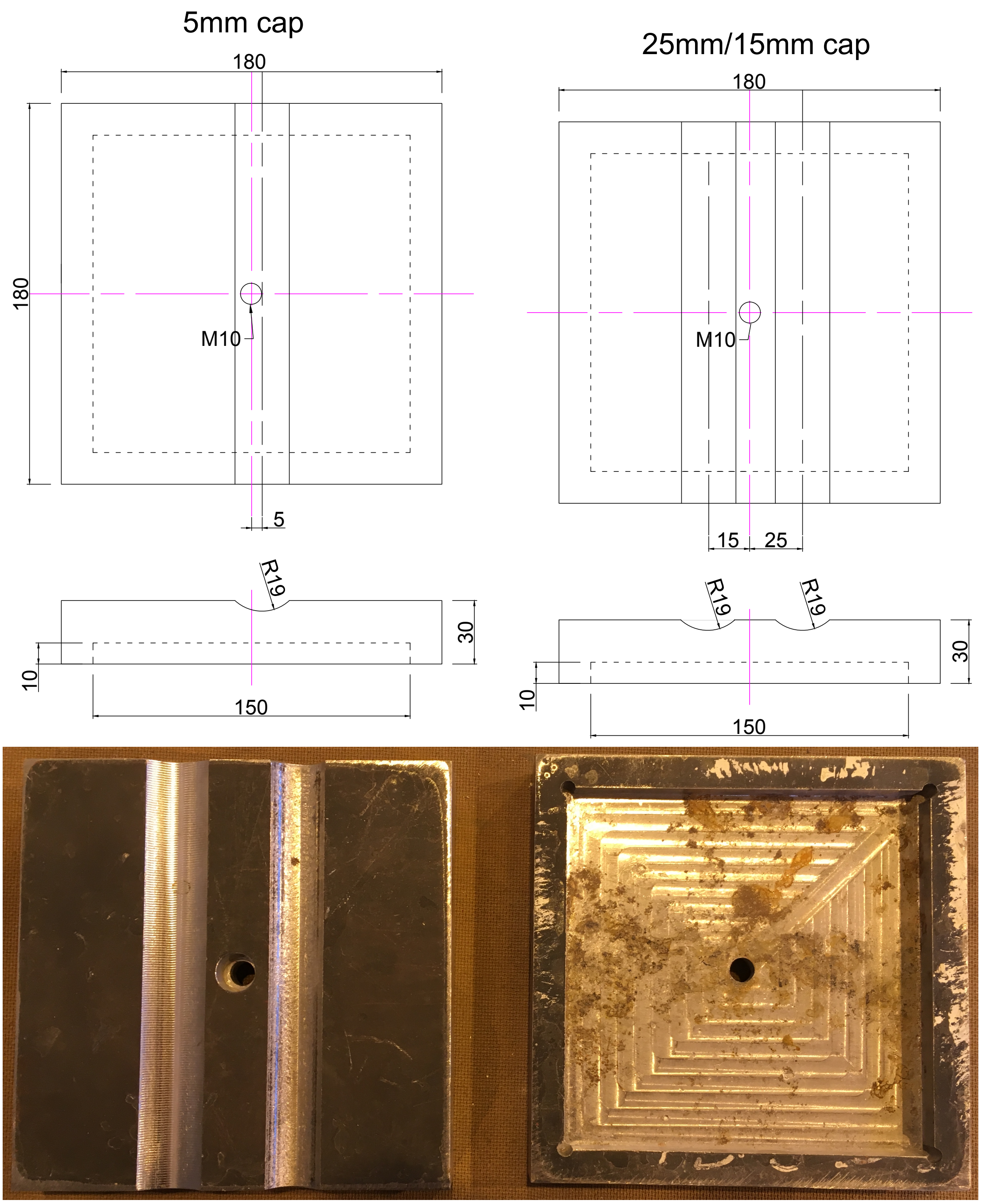


Figure 3.5: Steel plates for the top and bottom of columns

Two steel caps were manufactured to apply the specified load eccentricities. As the caps slipped onto the end plates of the columns, the same caps could be used multiple times, this helped to ensure that the load was applied at the same eccentricity for every column tested (within a given eccentricity grouping).

3.2.3 Thermocouples

As the columns were exposed to elevated temperatures during the experiments it was crucial that the temperatures within the column were measured at a sufficient number of locations to record an accurate spatial distribution of how the concrete was heated (and therefore potentially damaged), and to quantify the temperature gradients throughout the heating and cooling phases of the experiments as a result of the differing heat flux exposures used. This could then form the basis of theoretical work to predict the response, capacity, and fire damage resulting from the exposure to elevated temperatures. See Figure 3.7 for a schematic of the thermocouples (TCs) cast within the concrete columns.

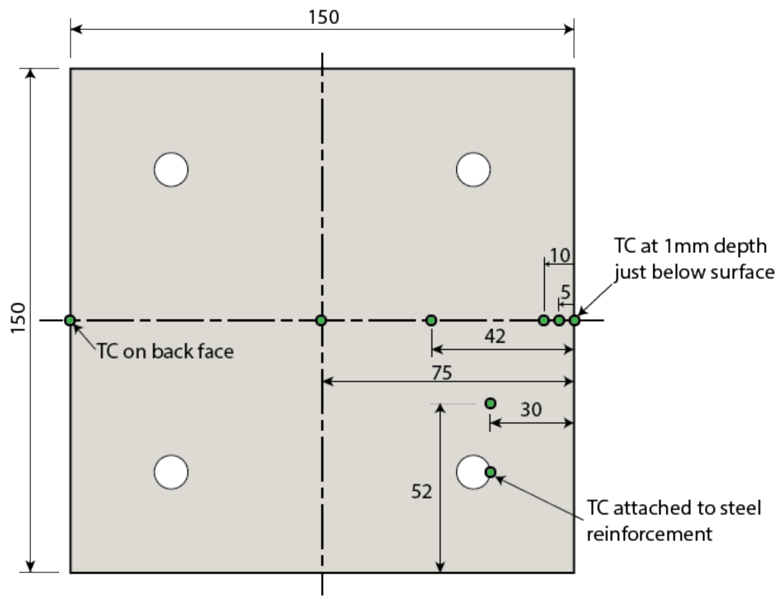


Figure 3.6: Plan of the location of thermocouples (TC) cast into concrete columns

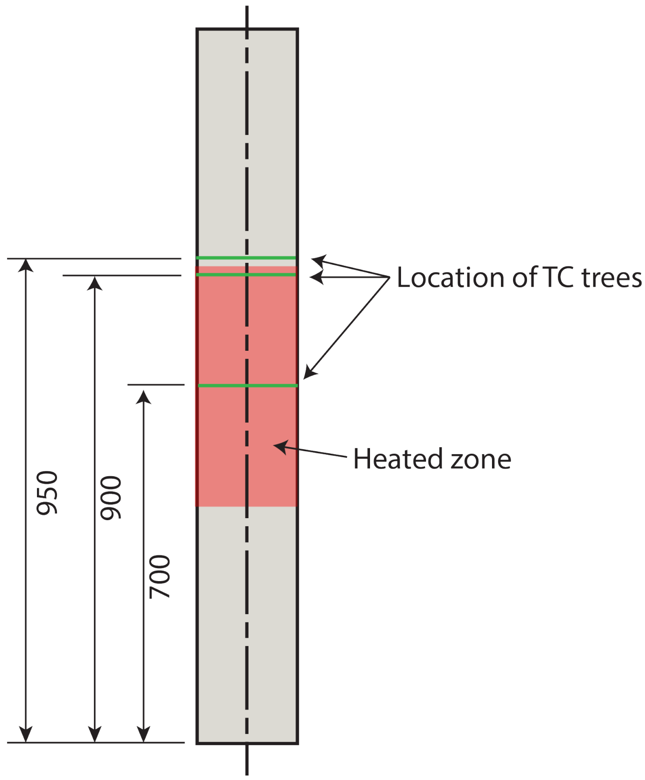


Figure 3.7: Elevation of the location of thermocouples cast into concrete columns

Type K thermocouples were located at three different heights (cross-sections) within the columns; these were at 700mm from the base of the columns (i.e. at mid-height), 900mm from the base of the column (i.e. at the top edge of the heated zone), and 950mm from the base (i.e. slightly above the top edge of the heated zone).

One challenge in locating the thermocouples as detailed in Figure 3.7 is ensuring that they did not move during the casting process when concrete was being placed into the formwork. To ensure that the thermocouples remained in the correct locations during this process, they were tied together and cast into rigid assemblies using epoxy resin. This created a tree of thermocouples that was sufficiently stiff to withstand the casting process without experiencing significant deformation. In addition, the thermocouple trees were attached to the formwork using steel tie wire so as to ensure that they did not move during casting operations. The tie wire was then removed along with the formwork after initial curing of the concrete. Figure 3.8 shows how the thermocouples and thermocouple trees were located in the formwork prior to the concrete casting process.

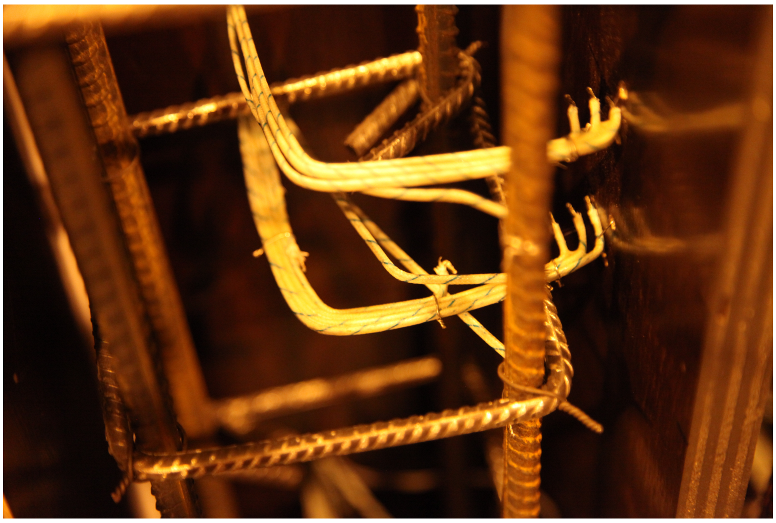


Figure 3.8: Thermocouples in formwork before concrete casting

Each TC tree was positioned within the concrete column as shown in Figure 3.7 and Figure 3.8. The TC trees were fed through the formwork and tied in place on the surface of the formwork using metal tie wire. To reduce the effect of the thermocouples on the reduction of the bending capacity of the columns, the wires of the thermocouples were fed to the centreline of the section and out the side of the columns. This maximised the area of concrete that could provide effective resistance when subjected to bending about a perpendicular axis (as in the tests performed in this thesis).

3.2.4 Strain Gauges

In testing the columns, it is useful to understand the contribution of the internal steel reinforcing bars to the total load sharing within the columns. This can be quantified using electrical resistance foil strain gauges, which were bonded to the steel reinforcement before casting. The strain gauges used were TML type FLA-5-11-3L strain gauges with a gauge length of 5mm. These were oriented in the longitudinal direction along the reinforcing steel bars at mid height.

As these strain gauges did not provide effective measurement of strain at elevated temperatures (indeed it is very difficult to undertake strain measurement with conventional electrical resistance foil gauges at elevated temperature under any circumstances), the data gathered by these can only be relied upon before the radiant panels were turned on. Thus, strain gauges were not applied to all concrete columns; only the control test specimens, which were tested to destruction at room temperature to determine the ambient strength of the columns under different load scenarios, were instrumented in this way.

The strain gauges were applied directly to the steel reinforcement at mid-height (i.e. 700mm from the base of the columns) so as to determine the strain in the reinforcement at the location of likely maximum strain - since these columns were tested in a pinned-pinned configuration and the maximum combination of axial-flexural loading is therefore expected at mid-height. In applying the strain gauges to the steel, the reinforcement was lightly ground and polished to a smooth finish to allow for application of the strain gauges. This occurred at the mid height of the reinforcement only with as small a footprint as possible so as not to significantly affect the performance of the bars in providing longitudinal reinforcement.

3.2.5 Deflections

Throughout the experimental test series presented in this thesis, the deflections of the columns were measured during every stage of the experiments from when the load was first applied, through heating and cooling, and when residually testing the strength of the columns that did not fail during heating or cooling. In order to collect this data both linear potentiometers (placed as shown in Figure 3.9) and digital image correlation (DIC) were used.

Linear Potentiometers

In order to capture the vertical and lateral deflections of the reinforced concrete columns during testing, it was possible (for most of the experiments, see below) to use linear potentiometers connected to a Vishay MicroMeasurements System 7000 data logger for data collection. Deflections were measured in four locations on the columns as shown in Figure 3.9, namely:

- at the base of the column measuring the stroke displacement of the hydraulic jack (vertical displacement at the base of the column)
- the lateral displacement at 350mm from the base
- the lateral displacement at 700mm from the base (mid-height), and;
- the lateral displacement at 1050mm from the base

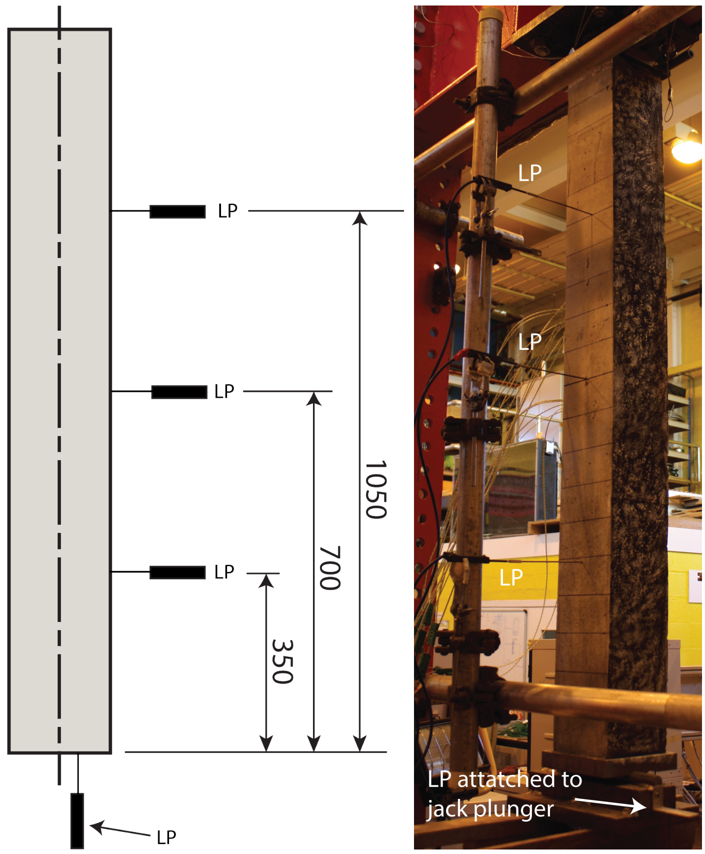


Figure 3.9: Location of linear potentiometers (LP) used to measure displacements

In addition to the above measurements, several linear potentiometers were placed on the loading frame to measure the relative displacement of the columns in relation to the loading frame itself. Most of the analysis completed as part of this thesis uses the data gathered by these linear potentiometers for assessing displacements during testing. However, due to the configuration of the experiments conducted on columns heated from both sides, it was not possible to use linear potentiometers for these tests. In place of this, digital image correlation (DIC) was used to determine the deflections of the columns.

Digital Image Correlation

In order to determine the deflections of the columns subjected to elevated temperature on both faces of the columns, the digital image correlation (DIC) software "GeoPIV-RG" was used [89]. Given a series of photos taken from the same location throughout the experiment, this software is capable of tracking selected patches of pixels in the images of the columns to determine the displacement. This technique is typically used when the photographs obtained are taken perpendicular the plane being measured (i.e. the plane being measured within photos is at a fixed distance and at right angles to the columns). However, due to the specific configuration of the test frame constructed, perpendicular photos that resulted in only in-plane photos could not be obtained in this specific scenario. Figure 3.10 details a typical photograph (field of view) used in the analysis of the columns using digital image correlation.

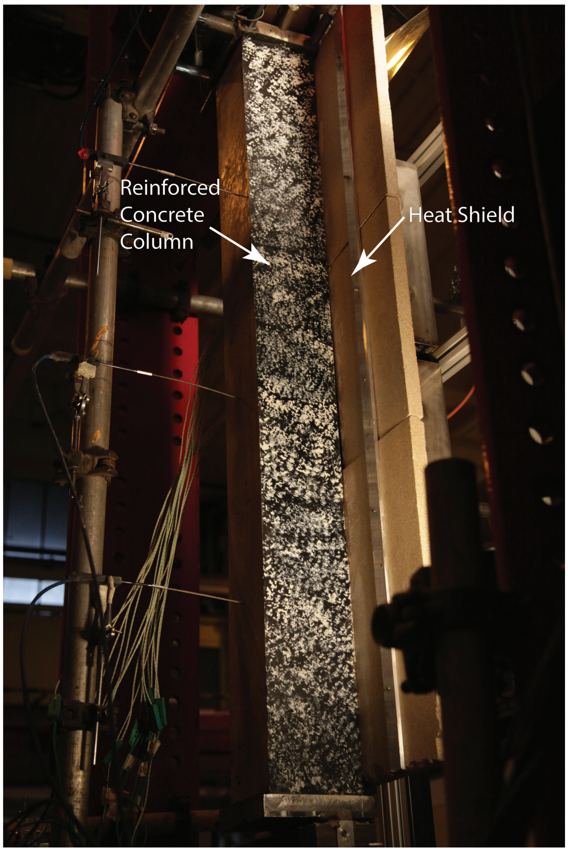


Figure 3.10: Typical photo used in the DIC analysis

It is clear from Figure 3.10 that the plane whose displacements are being measured (the black and white speckle-painted surface of the column) is not perpendicular to the camera. As a result, the DIC software can track the pixels and return the deflection of the columns in terms of pixels, however, as the photographs have been taken at an angle other than perpendicular, the true in-plane deflection cannot be easily ascertained. To convert the pixel values obtained from the DIC software into real in-plane measurements, additional MATLAB scripts were written to correct both for the angle of the photographs taken and for lens distortion of the camera. This was accomplished by using a using a checkered black and white board in one of the photographs positioned on the same plane as the surface being measured. Using this it was then possible to convert pixels into grid locations on the board and determine actual in-plane displacement. An example of the black and white board used in one of the columns has been detailed in Figure 3.11.

Figure 3.11: Checkered board used for DIC correction

The primary reason behind using digital image correlation was because measuring displacements for the specimens heated on both faces is difficult as linear potentiometers could not be used in this case. To validate that the technique used would provide sufficient accuracy to depend on the results when no linear potentiometers are present, the results were first compared with the displacements measured using the linear potentiometers. Figure 3.12 details the comparison of the LP data and the DIC data at midspan for sample 50-L60-E25-HF70-C-HL66. This sample was heated on the “compression face” and was exposed to a heated area of 66% of the column. This specimen failed almost immediately upon entering the cooling phase of the experiment at 145 minutes.

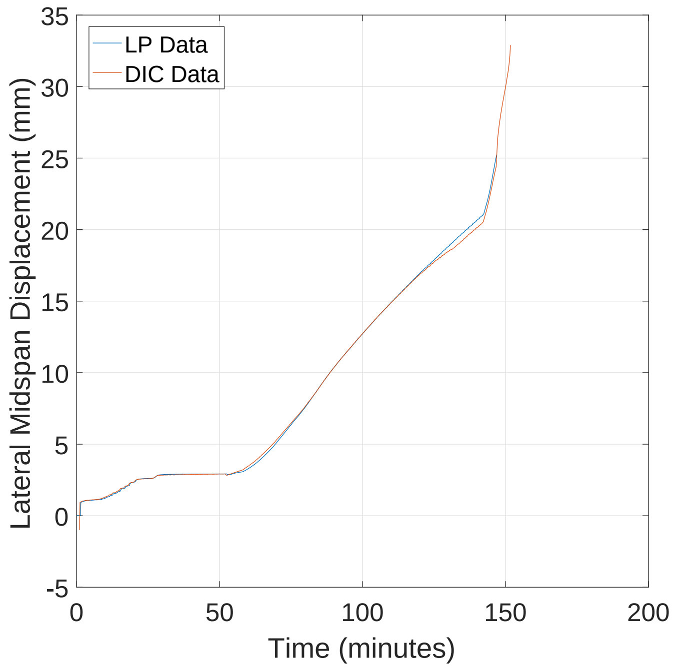


Figure 3.12: DIC correction validation

It is clear from Figure 3.12 that there was very good agreement between the LP data and the DIC data. The difference in the data tended to get slightly larger just before the column started to fail. This is true for all of the specimens. This is because the DIC software uses images to calculate the displacement in the columns. As the deflections in the columns increase and the concrete begins to crush, the plane that the DIC software relies on is becoming more distorted and cracked, making the pixels more difficult to track. When cracking and crushing of the face being measured becomes more and more evident, the accuracy in the DIC measurements decreased slightly. As the LP data and the DIC data was consistent throughout the test series, the "double heated" columns were measured using the DIC software only. Therefore, there is likely to be some error at the larger displacements in these tests but this error is unlikely to be of any consequence for the purpose of discussing the behaviour.

3.3 Test Matrix and Experimental Setup

This experimental test matrix used a total of 46 (two of the 48 samples cast had to be discarded due to honeycombing, as already noted) geometrically identical reinforced concrete columns to investigate the influence of the following parameters on their performance during the heating, cooling processes, and residually after cooling for 24 hours to ambient temperature:

- Magnitude of sustained axial compressive load applied;
- Eccentricity of the axial compressive load;
- Concrete compressive strength;
- Applied initial incident radiant heat flux;
- Number of side(s) simultaneously heated.
- Overall length of heated area.

The resulting overall test matrix is shown in Table 3.1 and Table 3.2, and the specifics of each of the varied parameters are described in the following sections.

3.3.1 Magnitude of Load Applied

The magnitude of the load applied to a concrete column during heating may not at first glance appear to be of great consequence in terms of influencing the post-fire response and performance of the structural element; however, as discussed in Chapter 2, concrete responds in a complex, stress and time dependant manner when exposed to thermal stresses and strains. The transient thermal creep phenomenon that occurs in concrete during heating may result in significantly different residual responses and capacities should this have any influence during heating. This is an issue that is not considered explicitly in the available design guidance [2]. As a result, two loading conditions were compared within the experimental programme so as to investigate their respective influence on the response of columns during all phases of loading and heating.

Load Case 1 (severe loading): The load was held at 60% of the columns nominal ambient capacity for the duration of the test.

Load Case 2 (mild loading): The load was held constant at 10kN ($\approx$ 1-2% of the columns ambient capacity) for the duration of the test

Load Application

For all of the tests conducted (both ambient tests to failure and sustained loading tests with heating), the axial compressive loading was applied using a 2000kN capacity hydraulic jack mounted within a self reacting structural steel frame. This jack was powered by computer-controlled hydraulic power pack with which the loading ramp could be specified in load control, or the load could be held constant for a specified period of time. This allowed for the load to be held constant during the heating and cooling stages of the experiments.

It should be noted that, in a test setup such as the one used in the current thesis, any thermal expansion of the concrete during heating would initially cause the concrete element to expand axially. This would in turn push back on the hydraulic jack supplying the load and, provided that the stroke of the jack remains constant, would result in a further increase in the load on the concrete column.

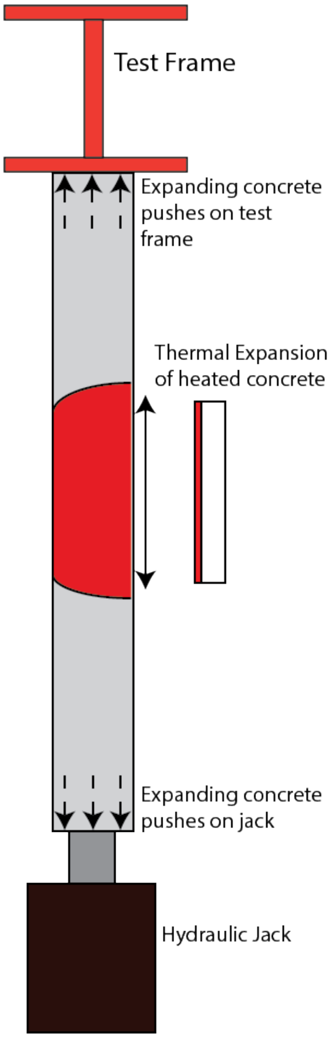


Figure 3.13: Increased load resulting from thermal expansion of concrete

In terms of a real, three-dimensional reinforced concrete structure, the thermal expansion of a column element would also result in an increase in the axial load (and thus stress) within the concrete; however, in practice this would depend on any movement at the column's end supports which would be determined by the stiffness of the surrounding structure (i.e. the degree of restraint provided by the cool surrounding structure). Assuming the surrounding structure is infinitely stiff against displacement (i.e. provides perfect restraint to thermal expansion), the column's end supports would not move despite the thermal expansion of the heated concrete, resulting in an increased stress state within the element itself. If, on the other hand, the surrounding structure has zero stiffness, the supports would deflect (move) and allow the element to expand freely. There would be no overall increase in the stress within the column due to restrained thermal expansion because there is no restraint provided by the surrounding structure (however there may still be differential thermal expansion and therefore differential thermal stresses that develop within the element itself).

Much the same consideration has to be made in the design of the experiments presented herein (see Figure 3.13). The increase in stress within the column due to the axial thermal expansion of the concrete in the test setup described would be a function of the stiffness of the frame and the hydraulic jack supplying the load. As this is a very difficult parameter to meaningfully quantify for the range of heating and loading conditions considered in the current experimental programme, the hydraulic power pack supplying the load was programmed to maintain the same hydraulic pressure throughout the experiment (once the loading hold point had been reached at ambient temperature). Therefore, as the columns expanded due to any increases in temperature (or contracted due to axial crushing or flexural effects), they pushed back on the hydraulic jack, thus momentarily increasing the load and the hydraulic pressure within the pump. The pump then released the excess pressure and jack retracted slightly to correct the load and supply the correct pressure.

This configuration therefore replicates the scenario of having a column which is restrained within a structure with effectively zero stiffness with respect to restraining axial deformations of the columns. During the cooling phase of the experiment, the opposite is true. The column will shorten as it cools, the pressure will decrease, and the pump will correct this by increasing the pressure and extending the jack.

The sustained, load control approach to loading described above was selected because, despite being unrealistic in terms of the likely restraint condition within a real structure (real structures can be expected to apply some axial restraint to a thermally expanding concrete column in practice) the loading methodology chosen at least provides a well characterised mechanical boundary condition - this was essential given that one of the primary motivations of the work presented in this thesis was to develop a suite of column test data that could be used in the future for credible validation of computational models for structural response to severe heating and cooling, as is badly needed in support of rational, performance-based structural fire design of reinforced concrete structural elements and structures.

3.3.2 Eccentricity of Load

The eccentricity of the applied axial compressive load is an important parameter due to the axial-flexural (and second order) effects that result under eccentric compressive loading. It will also ultimately determine the axial-flexural strain profile over the cross section when it is loaded. Thus, when changing the eccentricity of the compressive load applied to the column, it is possible to increase or decrease the gradient of the axial strain profile within the column, thus changing the respective magnitudes of tension and compression stresses applied at different locations within the column (see Section 4.6.5 for a more detailed discussion of this topic).

It is well established in the literature that the M-N interaction diagram for a square steel-reinforced concrete column can be calculated with a relatively high degree of certainty, particularly as to what combination of compressive load and bending moment any section can withstand at ambient temperature. However, as discussed in Chapter 2, it is considerably more difficult, due to the changing material properties of the concrete and reinforcing steel at elevated temperature, to reliably determine the M-N interaction diagrams for sections subjected to elevated temperatures.

Experimental Setup

To apply different degrees of initial axial load eccentricity, bespoke steel caps were manufactured to slip over the top and bottom of the columns, as previously discussed. These steel caps then allowed the load to be applied to the columns via a groove machined into it at a specified offset from the centreline. This ensured the same initial axial load eccentricity for all of the tests. Figure 3.5 provides details and schematics of the caps used for the application of the load at specific eccentricities. Figure 3.14 shows the interface between the steel caps and the loading platen attached to the hydraulic jack and the top of the self-reacting testing frame.

Figure 3.14: Interface between the loading plates and the steel caps attached to the columns

3.3.3 Incident Radiant Heat Flux

Effect of Heat Flux

The incident radiant heat flux is used to describe the thermal energy to which the concrete column is subjected per unit of time. Generally, when fire testing elements for structural fire resistance, this is done using a standard fire resistance testing furnace, typically following the ISO 834 gas phase temperature versus time curve [15] or similar [e.g. ASTM E119, BS 476]. This conventional method of fire testing, despite being the internationally applied standardised approach to assessing fire resistance experimentally, was intentionally not employed as part of the experimental test series presented herein. The heat flux to the specimens was instead applied using a propane-fired radiant panel, which could be placed at a specified distance from the target surface of the test specimen to achieve the desired incident radiant heat flux. This method of heating allowed for a severe, localised, non-uniform incident heat flux to be provided to generate a steep temperature gradient through the specimens (i.e. a temperature gradient which was representative of that which would be expected should the element be subjected on one face only to a ventilation controlled, cellulosic-fuelled, post-flashover compartment fire).

Experimental Setup

As previously stated, the initial incident radiant heat flux on the specimens was provided using gas fired radiant panels at a fixed offset distance from the target surface. The individual radiant panels themselves were 320mm x 420mm, and provided a constant incident heat flux on the surface of the specimen (see Section 3.3.3 for a discussion of the measured heat flux and the method of characterisation used).

The tests presented in this thesis aimed (in part) to investigate the effect(s) of non-uniform heating over specified areas of the columns, to compare the responses under different credible scenarios for a given concrete cross section. For the majority of the tests, only the middle third of the column was to be directly exposed to an incident radiant heat flux; the remainder of the column surfaces were not exposed to direct radiant heat fluxes originating from the radiant panels. To achieve this, radiation shields were manufactured from vermiculite panels and hung over the surfaces of the samples. This ensured that the samples were exposed to elevated temperatures only in the exposed areas through a window in the shield (see Figure 3.15). Therefore, the rise in temperature of the concrete in areas away from the heated surface area was dominated by conduction of thermal energy through the concrete itself, rather than radiation from exposure to the radiation from the panels (as well as convective losses from exposed surfaces).

This approach was taken so that the heated area of the column could be precisely controlled and characterised again because this is considered critical in carefully defining the thermal boundary conditions of the tested elements given that the collected data is intended to be used for detailed validation of structural fire engineering computational analysis packages in the future. Figure 3.15 shows details of the vermiculite shield (after multiple uses hence minor thermal damage).

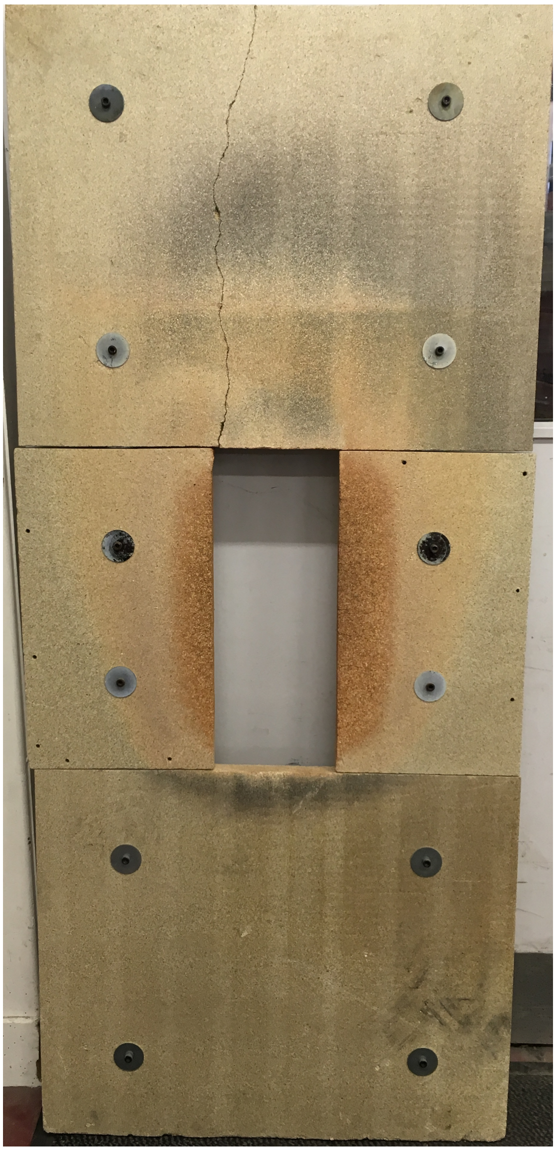
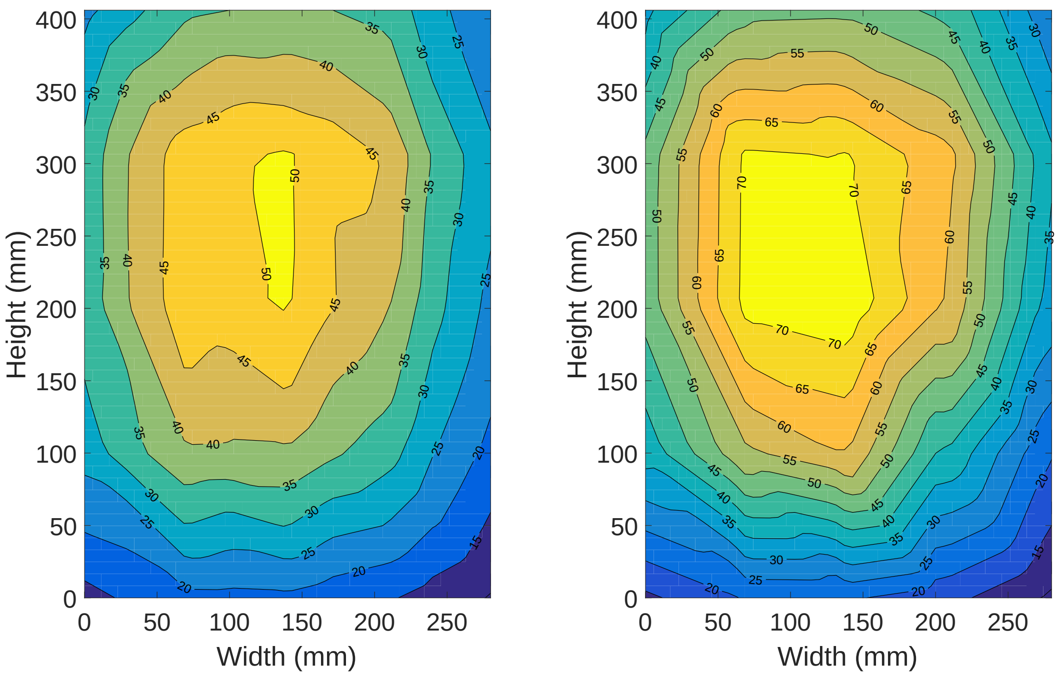


Figure 3.15: Shield used to expose only a defined area to the incident radiant heat flux induced by the radiant panels

Mapping the Incident Radiant Heat Flux

The distribution of incident heat flux over the exposed surface of the column was not constant over the entire face, given the variation in radiation view factor at any particular location on the heated surface and the interaction of the surface with convective currents generated both by the radiant panels themselves and by the hot surface of the column. The particular radiant panels used burn very consistently (i.e. at a constant surface temperature once a steady-state condition is reached (in about 30-60 seconds from being ignited) and proved to be very reliable in providing a temporally steady heat flux over a long period of time during testing.

The radiant panels used proved to be very repeatable throughout the entire test series, any repeat tests reproduced almost identical behaviour. It should however be noted that there are a number of caveats with regard to this method of imposing the thermal boundary condition. To map the incident heat flux over the entire area of the radiant panels, a water cooled heat flux gauge (Schmidt Boelter gauge) mounted on a robotic arm was used. Measurements were taken in a grid pattern at 25mm by 50mm. Five measurements were obtained at each point and the average taken. From this it is possible to obtain a map of the incident heat flux over the surface of the concrete when it is placed at a specified distance from the radiant panel. For the experiments conducted, the maximum incident radiant heat fluxes to which the columns were exposed were 50kW/m^2 and 70kW/m^2; these correspond to the heat fluxes at 100mm and 75mm distances from the surface of the panel to the target surface, respectively. The heat flux maps obtained at these distances over the area of the panels is detailed in Figure 3.16.



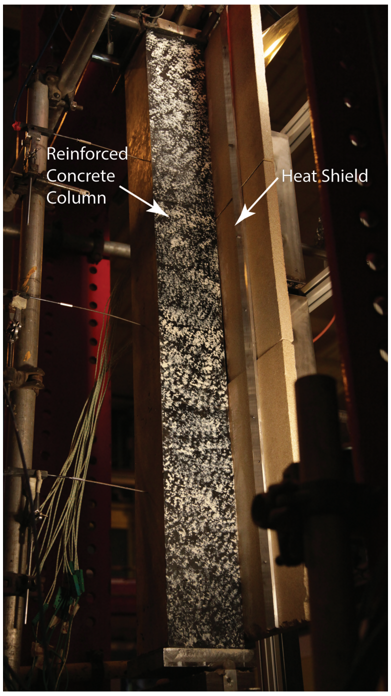
(a) Heat flux at 100mm from surface (kW/m^2)

(b) Heat flux at 75mm from surface (kW/m^2)

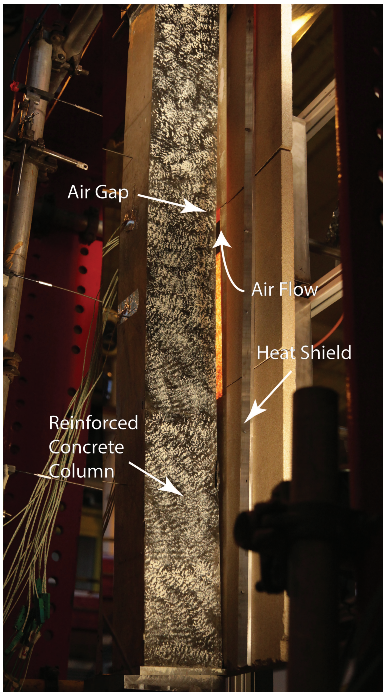
Figure 3.16: Incident heat flux map of the radiant panel used to expose columns to elevated temperature

It is clear that the heat flux is not uniform over the surface of the exposed area of the columns. This was influenced by the fact that the panels did not burn in an entirely uniform fashion i.e. the panels were slightly hotter left of centre, as can be seen in Figure 3.16. In addition to this, the heat fluxes that were desired to damage the concrete columns were located close to the surface of the radiant panels, where convection will also play a role in heating the surface of the columns. This in turn explains why the area of maximum heat flux on both of the heat maps provided in Figure 3.16 are located slightly above the centreline.

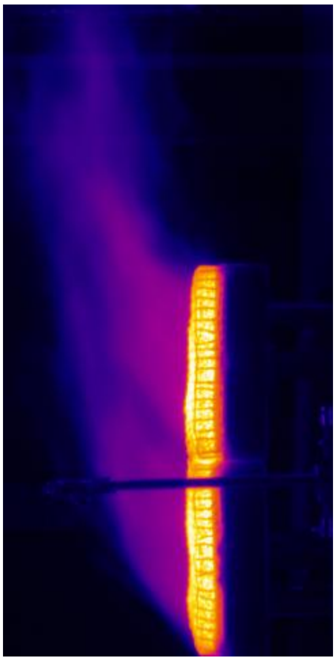
The columns were protected by a shield with a predetermined exposed area, see Section 3.3.3, to ensure that the surface of the column outside the exposed area was not subjected to any direct heat flux from the radiant panels. It was however, noticed that during certain experiments, specifically those heated on the compression face, the shield deflected away from the face of the panel in a concave manner, in addition to the column itself deflecting. This opened up a small air gap between the shield and the face of the concrete, allowing convective flow up the face of the concrete column. This would have therefore allowed slight heating of the surface of the concrete column that should have been protected by the shields.



(a) Test setup at the beginning of the experiments before the heating phase has begun



(b) Convective flow of air during the heating phase of the experiments



(c) Thermal imaging of the convective flow of gasses from the surface of the radiant panel.

Figure 3.17: Illustration of the influence of convective air flow during the experiments

The role of convection is not limited to increasing the heat flux above the centre of the column, the nature of the way that the incident heat flux was measured raises a number of questions about accuracy of the calibrations conducted in this section. These errors and the methods taken to minimise their influence on the validity of the experimental results have been detailed in Section 3.3.4.

3.3.4 Heat Flux Calibration Error

The method of calibrating the heat fluxes from the radiant panels has been described in Section 3.3.3. A Schmidt Boelter heat flux gauge was used. This is an instrument that calculates heat flux by measuring the increase in temperature of the water cooled surface of the gauge. This in an important distinction to make because this method of measuring the heat flux will influence the incident heat flux that is received by a sample when it replaces the heat flux gauge. This has been illustrated by exploring the nature of the heat transfer equation for the heat flux gauge and the concrete samples respectively.

Energy Equation

When considering the transfer of energy into the heat flux gauge and a concrete column in this particular case (close to the surface of the panels), both conduction and radiation will play a role. The energy equation for this mode of heat transfer is therefore detailed in Equation 3.1.

$$\dot{q}''_{total} = \dot{q}''_{rad} + \dot{q}''_{conv} \tag{3.1a}$$

$$\dot{q}''_{rad} = \sigma\varepsilon(T_p^4 - T_s^4) \tag{3.1b}$$

$$\dot{q}''_{conv} = h_c(T_g - T_s) \tag{3.1c}$$

Where:

$\dot{q}''_{total}$ = Total heat flux (W/m^2)

$\dot{q}''_{rad}$ = Radiative heat flux (W/m^2)

$\dot{q}''_{conv}$ = Convective heat flux (W/m^2)

σ = Stephan boltzmann constant $(W.m.K^{-4})$

ε = Emissivity (W/m^2)

T_p = Temperature of the radiant panel (K)

T_g = Temperature of the gas (K)

T_s = Temperature of the surface (K)

Heat Flux Gauge

As previously detailed, the surface of the heat flux gauge is water cooled and therefore forced to maintain a lower temperature (i.e. $T_s \approx 20^oC$). Considering Equation 3.1, if the gauge was located outside the convective field of the radiant panel, the temperature of the gas would be low ($T_g \approx 20^oC$) similar to the temperature of the surface of the heat flux gauge. Convection would therefore not play a very large role in the equation which means that the heat flux measured would relate only to the incident radiant heat flux on the surface. This however changes when the heat flux gauge is moved closer to the surface of the panel and the temperature of the gas rises. This will result in a larger difference between the temperature of the gas and the temperature of the surface of the gauge. The measured value in this case will then relate to both the incident radiant heat flux and the convective heat flux on the surface of the gauge.

Concrete Specimen

When a concrete specimen is placed in the test rig as opposed to the heat flux gauge, the largest noticeable difference is that the concrete specimen is not water cooled. Therefore, it may be assumed that the surface of the concrete rises to match the conditions of the surrounding environment. In the case where the concrete specimen is located close to the surface of the panel, the energy equation will change every time step depending on the temperature of the surface of the concrete. In the beginning, when the concrete is at a low temperature, convection will play a large role in Equation 3.1 because of the large difference in the temperature of the gas and the surface temperature of the concrete. As the temperature of the concrete increases however, the difference in the two temperatures will reduce dramatically, reducing the influence of convection in the heat transfer equation.

If the calibration from the heat flux gauge related only to radiation, this can be accounted for in any heat transfer analysis because the heat flux would relate only to the incident radiant heat flux and convection would not need to be accounted for. In this case however, the calibration includes both radiation and convection. As the influence of convection changes throughout the experiment when a concrete specimen is in place, it is evident that the numbers obtained in the calibration will not relate directly to the actual heat flux that the columns are exposed to.

It can therefore be concluded that the thermal boundary condition maintained throughout this experimental test series are not entirely characterised by the method of calibration used. This does not however invalidate the discussion and conclusions drawn from this experimental test series. As a result of this error in the calibration, many thermocouples were cast into the concrete columns tested because it is ultimately the internal temperatures of the concrete columns that will determine their behaviour. Because the internal temperature of the concrete are known, quantitative analysis of the response of the concrete columns is still very much possible.

3.3.5 Face(s) Heated

In addition to varying the maximum initial incident radiant heat flux on the exposed surface of the column, the number of faces of the column heated was also varied between experiments. The faces heated in the current study were the front and the back of the column only; the sides were never heated directly so that bi-axial bending moments were not induced during the experiments. This was important so as to reduce the level of complexity in the response (and in any subsequent analysis undertaken within the current thesis or by others).

Definitions of face(s) heated

Throughout this thesis reference is made to the face heated when describing the experiments conducted and the analysis undertaken. For the sake of simplicity, the heated face of the column is be referred to as the "compression face" or the "tension face". It is of course important to note that these names do not necessarily describe the stress state within the columns at these faces. The stresses within a column that is tested with a 5mm load eccentricity are consistently compressive, and tensile stresses are not present since the load is applied well within the kern (i.e. middle third) of the cross section. In this case the “less compressed” face is referred to as the “tension face”, and the “most compressed” face is referred to as the “compression face”. Thus the “tension face” and the “compression face” of the columns are referenced throughout. This definition has been used to help the reader immediately recognise the direction of bending of the column - it does not necessarily describe the actual stress state at the surface of the face in question.

3.3.6 Heated Length

The final parameter varied as part of the experimental programme was the length of the heated region of the columns. As discussed previously, the approach normally taken in standard structural fire resistance testing is to expose the entire length of an element to elevated temperatures (or at least the longest practicable length, which is typically greater than 70-80% of the total length of the element). As part of experimental programme in this thesis the length of the area exposed to incident radiant heat flux was varied so as to observe any difference in response in heating, cooling, or residually.

Varying the length of the heated face raises the opportunity to observe a number of interesting responses of the column. This is because it is not simply possible to conclude that, as the concrete gets hotter it loses strength therefore, a larger heated area is worse. Concrete is a complex material, and it is conceivable that under certain loading scenarios, a larger heated area could benefit the mechanical response of the element during the heating phase. Particularly in the case of a “compression heated” column versus a “tension heated” column.

Experimental Setup

As discussed in Section 3.3.5, a vermiculite shield was used to shield most of the column from radiant heating from the radiant panels, and to ensure that only a well defined region of the column was exposed to direct radiant heating. Two separate shields were manufactured:

1. a shield with a 465mm (tall) by 150mm (wide) window to expose the middle third of the columns and;
2. a shield with a 930mm (tall) by 150mm (wide) window to expose the middle two thirds of the columns.

The shield used for the experiments with a smaller exposed area is shown in Figure 3.15.

3.3.7 Total Experimental Assembly

So far in Section 3.3 the parameters investigated as part of the experimental programme, and the design decisions/methods used to achieve this end, have been discussed. The testing rig was designed and manufactured in a modular fashion so that it could be modified as required to investigate the different parameters discussed in this chapter.

Support Conditions

All columns were pin supported at both ends during testing. This was achieved, as already discussed, by using steel end caps machined to slip over the top and bottom integrated steel end plates of the columns, with semi-circular grooves machined at the requisite different load eccentricities noted in Table 3.1 and Table 3.2. The purpose of the steel caps was to:

- enable fully pinned end conditions (assuming negligible friction in the pins) at the top and bottom of the columns;
- ensure that each of the columns was loaded to the same level of initial axial load eccentricity for each of the test cases; and
- enable easy removal and reinsertion of the columns to allow them to be replaced quickly so that testing could progress as efficiently as possible (given the large number of specimens tested).

Ultimately this method of applying the load resulted in an easy, reliable and consistent method of applying a specified load at a certain eccentricity to the columns. Section 3.2.2 details the design of the steel caps.

The load was applied using the same 2000kN hydraulic jack, programmed to the same settings (aside from variations in the value of the load hold), for all sustained loading tests.

Heat Flux

As detailed in Section 3.3.3, the heat flux was applied using gas fired radiant panels attached to an aluminium frame that could be easily moved to any required position and distance from the target surface of the columns, thus allowing for a range of heat fluxes, heated lengths, and number of heated surfaces to be investigated. Each of the vermiculite radiation shields used during the experiments was hung from the top of the column in such a way that it could deflect and move with the columns as they deformed during testing, so as to ensure that their integrity was not compromised at any point throughout the experiments due to mechanical or differential thermal (i.e. bowing) deformations.

The full test setup is shown in Figure 3.18.

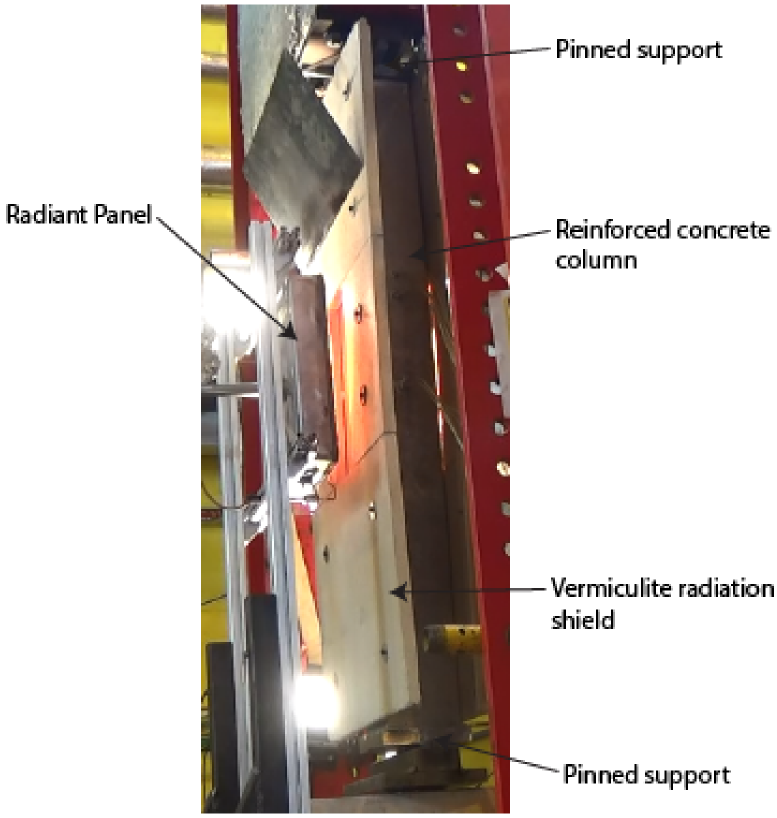


Figure 3.18: Full test setup shown during testing of specimen 50-L60-E5-HF70-T-HL33

3.3.8 Discussion of Parameters Investigated

Section 3.3 has set out the experimental setup and the basis of the parameters investigated in this thesis. To summarise, a total of 46 experiments have been undertaken under a range of different loading and heating conditions. The purpose is to investigate various different external factors that may affect the response and performance of the columns during heating, cooling, and residually.

A range of additional factors could affect the response and performance of reinforced concrete columns subjected to severe heating; some of these include:

- concrete mix proportions;
- curing conditions;
- stress distribution throughout the cross section;
- support conditions;
- temperature gradients through the section;
- concrete cover to the internal steel reinforcement;
- bi-axial versus uni-axial bending configurations;
- the number of faces heated, potentially the side faces as well a the front and back faces;
- the heated length;
- the cooling conditions (i.e. rate and manner);
- etc, etc...

Considering this non-exhaustive list, it is clear that volume of work and experimentation required to investigate all of these parameters would be considerable. However, it is important to note that, no matter what the external influences on a column are, the two most important parameters which ultimately determine the response of a reinforced concrete column are likely to be:

1. the stress distribution within the section (due to mechanical and/or thermal effects); and
2. the temperature gradient throughout the element (during heating and cooling).

No matter the number of **external** influences on the column, the strength and mechanical performance will ultimately be governed by these two internal parameters. This test series, despite changing a large number of external parameters, has changed those parameters in a methodical and controlled fashion ultimately, to investigate the influence of the two internal parameters stated above (stress state and thermal gradients).

Throughout the remaining sections there will be reference to the external factors influencing the column but the basis of the investigation, and ultimately the conclusions, will be based upon the effect of the stress distribution and temperature gradient on the response of the specimens. As measurements of both, the external boundary conditions and the internal temperatures have been measured, this experimental approach therefore enables the interrogation of the following two areas of the state of the art:

1. The ability of the current guidance, for response of concrete at elevated temperatures, to predict the internal conditions of the columns based on the external influences.
2. The ability of the current guidance for response of concrete at elevated temperatures to predict the physical response of the elements based on the internal conditions within the elements during heating, cooling, and residually.

3.4 Experimental Procedure

As detailed in Section 3.1, the tests themselves take place in five distinct steps (these are described in detail in the following sections):

Step 1: Apply the load to the column;

Step 2: Heat the column for a specified duration (unless a control test at ambient temperature is performed, in which case only Step 1 is relevant);

Step 3: Allow the column to cool to ambient (unless the column failed during heating, in which case only steps 1 and 2 are relevant);

Step 4: Conduct a non-destructive damage assessment of the column (unless the column failed during cooling, in which case only steps 1-3 are relevant); and

Step 5: Destructively test the column to failure to determine its residual (post-heating) structural response and capacity.

Step 1 - Apply the load

The first step in the tests was to apply the load to the columns. This step was automated by the hydraulic power pack controlling the pressure within the hydraulic jack. The power pack was programmed to increase the load at a rate of 15kN/minute until the target load was reached. Once the target load was reached, the load was maintained constant for the remainder of the test or until the pump is switched off.

If the test being conducted was an ambient temperature control test, the load was ramped at a rate of 15kN/minute until the column reached its capacity and failed (i.e. no heating was performed in these cases).

Step 2 - Heat the column

Once the target load had been reached, the load was held constant for a period of 20 minutes, to allow the column to seat and settle, and the deflections to stabilise before heating. After this 20-minute period had elapsed, the radiant panels, which were positioned in front (and behind where relevant for two cases with two faces heated) of the column, were switched on. The column was then subjected to 90 minutes of heating by a constant incident radiant heat flux while the sustained load that applied in Step 1 remained constant, regardless of thermal or mechanical deformations of the column (i.e. load control mode at a constant load was used).

Step 3 - Cool the column

At the end of the 90-minute heating period the radiant panels are switched off and removed. Importantly, and unlike the vast majority of previously performed research on the fire performance of reinforced concrete columns, this allowed the specimen to cool whilst being subjected to the sustained load applied in Step 1. The response of the columns during heating was measured and recorded continuously. The columns were left to cool in the ambient air within the Structures Laboratory at the University of Edinburgh, without any forced cooling. After a period of four hours of cooling the pump was switched off and the load removed from the column. The specimen was then left unloaded for 24 hours before being loaded to failure at ambient temperature.

Step 4 - Conduct a "damage" assessment

Twenty-four hours after the load was removed from the column, a non-destructive damage assessment was undertaken on all columns that had not failed during heating or cooling using the ultrasonic pulse velocity test method. Measurements were taken over a grid over the surface of the column to map the damage caused by the radiant panel. The specific method used, and its outcomes, are described in Chapter 5.

Step 5 - Destructively test the column

Finally, after the damage assessment, the columns that had not failed during ambient testing, heating, or cooling, were tested to failure under the same support conditions as they were during heating and cooling. The hydraulic power pack was switched back on and the load was increased again at a rate of 15kN/minute until the column reached its capacity and failed.

3.5 Test Series

Given the experimental method and test setup described in the previous sections, Table 3.1 and Table 3.2 detail all of the reinforced concrete columns and the conditions they were exposed to during the experiment. The tests were referenced in the following manner:

Specimen name: 30-L60-E5-HF50-T-HL33

This naming convention should be read as:

30MPa concrete - 60% axial load capacity - 5mm load eccentricity - 50kW/m^2 heat flux - heated on the "tension face" (C for "compression face") - 33% heated length of the column

Table 3.1: Experiments conducted on concrete columns constructed using 30MPa concrete

Test Reference	Concrete Strength (MPa)	Load (%)	Load eccentricity (mm)	Incident Heat Flux (kW/m^2)	Face(s) Heated (C/T)
30-L100-E5-HF0-NA-HL0-(1)	30	100	5	0	N/A
30-L100-E5-HF0-NA-HL0-(2)	30	100	5	0	N/A
30-L100-E15-HF0-NA-HL0-(1)	30	100	15	0	N/A
30-L100-E15-HF0-NA-HL0-(2)	30	100	15	0	N/A
30-L100-E25-HF0-NA-HL0-(1)	30	100	25	0	N/A
30-L100-E25-HF0-NA-HL0-(2)	30	100	25	0	N/A
30-L60-E25-HF50-T-HL33	30	60	25	50	T - 33%
30-L60-E25-HF50-C-HL33	30	60	25	50	C - 33%
30-L60-E5-HF50-C-HL33	30	60	5	50	C - 33%
30-L60-E5-HF50-T-HL33	30	60	5	50	T - 33%
30-L60-E25-HF70-T-HL33	30	60	25	70	T - 33%
30-L60-E25-HF70-C-HL33	30	60	25	70	C - 33%
30-L60-E15-HF70-T-HL33	30	60	15	70	T - 33%
30-L60-E5-HF70-C-HL33	30	60	5	70	C - 33%
30-L60-E5-HF70-T-HL33	30	60	5	70	T - 33%
30-L1-E25-HF70-C-HL33-(1)	30	1	25	70	C - 33%
30-L1-E25-HF70-C-HL33-(2)	30	1	25	70	C - 33%
30-L1-E25-HF70-T-HL33	30	1	25	70	T - 33%
30-L1-E5-HF70-C-HL33	30	1	5	70	C - 33%
30-L1-E5-HF70-T-HL33	30	1	5	70	T - 33%
30-L60-E25-HF70-CT-HL33	30	60	25	70	C+T - 33%
30-L60-E5-HF70-CT-HL33	30	60	5	70	C+T - 33%
30-L1-E25-HF70-CT-HL33	30	1	25	70	C+T - 33%
30-L1-E5-HF70-CT-HL33	30	1	5	70	C+T - 33%

Table 3.2: Experiments conducted on concrete columns constructed using 50MPa concrete

Test Reference	Concrete Strength (MPa)	Load (%)	Load eccentricity (mm)	Incident Heat Flux (kW/m^2)	Face(s) Heated (C/T)
50-L100-E25-HF0-NA-HL0-(1)	50	100	25	0	N/A
50-L100-E25-HF0-NA-HL0-(2)	50	100	25	0	N/A
50-L100-E5-HF0-NA-HL0-(1)	50	100	5	0	N/A
50-L100-E5-HF0-NA-HL0-(2)	50	100	5	0	N/A
50-L60-E25-HF70-C-HL33-(1)	50	60	25	70	C - 33%
50-L60-E25-HF70-C-HL33-(2)	50	60	25	70	C - 33%
50-L60-E25-HF70-T-HL33	50	60	25	70	T - 33%
50-L60-E5-HF70-T-HL33	50	60	5	70	T - 33%
50-L60-E5-HF70-C-HL33	50	60	5	70	C - 33%
50-L1-E5-HF70-C-HL33	50	1	5	70	C - 33%
50-L1-E5-HF70-T-HL33	50	1	5	70	T - 33%
50-L1-E25-HF70-C-HL33	50	1	25	70	C - 33%
50-L1-E25-HF70-T-HL33	50	1	25	70	T - 33%
50-L60-E25-HF70-CT-HL33	50	60	25	70	C+T - 33%
50-L1-E25-HF70-CT-HL33	50	1	25	70	C+T - 33%
50-L60-E5-HF70-CT-HL33	50	60	5	70	C+T - 33%
50-L1-E5-HF70-CT-HL33	50	1	5	70	C+T - 33%
50-L60-E25-HF70-C-HL66	50	60	25	70	C - 66%
50-L60-E25-HF70-T-HL66	50	60	25	70	T - 66%
50-L60-E5-HF70-C-HL66-(1)	50	60	5	70	C - 66%
50-L60-E5-HF70-C-HL66-(2)	50	60	5	70	C - 66%
50-L60-E5-HF70-T-HL66	50	60	5	70	T - 66%

Chapter 4

Experimental Results: Heating and Cooling

4.1 Introduction

In Chapter 3, an outline of the experimental program conducted during this PhD project has been provided, along with a description of the experimental apparatus, methods employed, specimens cast, and testing parameters varied. This chapter presents some of the results of these experiments with particular emphasis on the structural response of the columns during the heating and cooling phases only. As noted in Chapter 3, post-heating residual experiments were conducted on all specimens that did not fail during the heating/cooling phases of the experiment. Chapter 5 presents results from these residual experiments. The results presented will specifically concentrate on the raw data collected from each experiment (deflection, temperatures etc.), for analysis of the experiments and a comparison to the sectional analysis model created, see Chapter 6.

The ambient temperature response of concrete columns has been researched extensively (see Chapter 2), therefore the results from the ambient temperature control experiments are presented purely as a baseline for understanding the performance of the columns subjected to elevated temperatures.

4.2 Ambient Temperature Control (Benchmark) Experiments

As stated in Section 3.5, 46 experiments were conducted on concrete columns. However, only five combinations of loading conditions were considered throughout the entirety of the experimental program in this thesis (60% load - 5mm, 15mm and 25mm load eccentricity and 10kN load - 5mm and 25mm load eccentricity). Each of the 36 heated experiments conducted is a repeat of one of the five loading conditions with differing levels of damage induced by a radiant heat flux. The conditions of the ambient temperature experiments conducted have been detailed in Table 4.1. Each load combination was tested to failure under ambient temperature conditions in duplicate. These experiments were load controlled and allow for the characterisation of the load deflection path taken by the columns under ambient temperature conditions.

As described in Section 3.4, the experimental procedure for the heated experiments was to apply and hold the axial compressive load at a specified level while exposing the columns to elevated temperatures. For the heated column experiments, the ambient temperature response of the columns can only be compared until the sustained load level for that experiment is reached (i.e. 60% of the ambient temperature capacity of the column for the heavily loaded columns and, realistically not at all for the lightly loaded columns). These pre-comparisons provide further checks on both the consistency and repeatability of the experiment setup and the columns' responses.

4.2.1 Load Deflection Responses

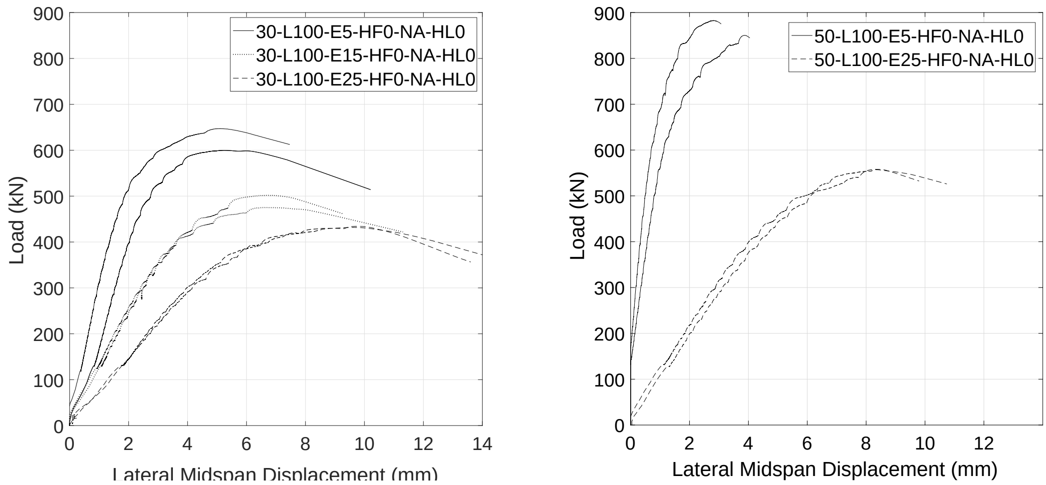
Given the experimental program designed, the first step in understanding the responses of the columns at elevated temperature is to determine their ambient temperature responses. Many of the parameters investigated in this thesis involved changing the heating conditions or the magnitude of the applied compressive load that was sustained during heating. The only parameters that were changed during the experimental program that would affect the ambient temperature response of the columns prior to heating were; (1) the concrete strength, and (2) the loading eccentricity.

Table 4.1: Selected results from ambient temperature experiments

Specimen identifier	Concrete Strength (MPa)	Load eccentricity (mm)	Ultimate Load (kN)	% difference
30-L100-E5-HF0-NA-HL0	30	5	647.21	7.8
30-L100-E5-HF0-NA-HL0		5	599.77	
30-L100-E15-HF0-NA-HL0		15	501.72	5.3
30-L100-E15-HF0-NA-HL0		15	475.07	
30-L100-E25-HF0-NA-HL0		25	431.90	0.8
30-L100-E25-HF0-NA-HL0		25	434.35	
50-L100-E5-HF0-NA-HL0	50	5	850.31	3.8
50-L100-E5-HF0-NA-HL0		5	882.63	
50-L100-E25-HF0-NA-HL0		25	558.01	0.1
50-L100-E25-HF0-NA-HL0		25	557.08	

It is noteworthy in Table 4.1 that eccentricities of 5mm, 15mm, and 25mm were investigated for the lower strength (30MPa) concrete columns, whilst only 5 and 25mm were investigated for the higher strength (50MPa) columns. The reason for this was because of an interesting deflection history that was observed during the heated experiments on the lower strength columns. This aspect of the experiments is discussed in more detail later in Section 4.6.

The two primary objectives of the ambient temperature loading tests were (1) to determine the ambient temperature axial-flexural load-deflection responses of the columns, and (2) to ensure that the experiment rig designed could provide reasonably repeatable testing conditions. It is clear from the ultimate load values given in Table 4.1 and the load-deflection response graphs in Figure 4.1 that the duplicate experiments demonstrated good repeatability (within 1% for the ultimate load for the 25mm eccentricity columns and 7% for the 25mm eccentricity columns). In particular, the experiments on columns with a 25mm load eccentricity were highly consistent; as should be expected given that these tests are least sensitive to small instabilities and column imperfections.



(a) C30 Concrete **(b)** C50 Concrete

Figure 4.1: Ambient load-midspan deflection for (a) C30 and (b) C50 concrete columns

As can be expected from the ambient temperature tests conducted, the following conclusions can be drawn from Figure 4.1:

1. the stronger the concrete, the higher the ultimate load for the corresponding load eccentricity;
2. the stronger the concrete, the stiffer the column; and
3. the higher the load eccentricity, the smaller the ultimate load carried as a result of the increased moment induced within the column.

There is nothing surprising about these observations as they are expected on the basis that stronger concrete produces a stronger and stiffer column. However, the most important point to take from Figure 4.1 is that the test setup detailed in Chapter 3 can produce load-deflection responses that follow similar paths when applying the same eccentricity every test. This is an important point, as all of the further tests conducted are to be compared directly with one of the five ambient temperature test conditions detailed in Table 4.1 and Figure 4.1.

4.2.2 Failure Modes

The failure modes displayed in the ambient temperature experiments for the reinforced concrete columns were consistent for all 10 specimens. Upon loading, each column shortened and deflected laterally (bowed) as a result of the load eccentricities applied. The severity of lateral deflection was dependent on the strength of the concrete from which the column was fabricated and the eccentricity of the load applied, as illustrated in Figure 4.1. All columns failed in compressive bending, where the concrete on the more compressed side (i.e. compression side) of the column crushed and horizontal cracks appeared on the least compressed (i.e. tension) side. Typical photos of these failure modes are shown in Figure 4.2.

Figure 4.2: Typical failure mechanisms of reinforced concrete columns

As can be seen in Figure 4.2, all of the columns failed in a comparatively similar fashion i.e. every column failed in bending under axial-flexural loading. The location of the point of failure was, however, not easily predictable. It would be expected in a setup such as that used in the current study (i.e. a symmetrical column pin supported at both ends) that the point of failure would be at the mid-height of the column on its most compressed face. This slight inconsistency in the tests presented herein may be due to the inherent variability of concrete or differences in the spacing of the steel ties through the depth of the column. It is, however, clear from Figure 4.1 detailing the load-deflection of the columns, that this did not appear to have had a significant affect on the strength or deflection histories displayed by the columns during the tests. Therefore, despite failing in slightly different locations during the ambient temperature tests, the columns displayed similar behaviour in all of the loading configurations tested. The direction of failure in Figure 4.2 (towards or away from the camera) was purely a function of the location of the load eccentricity, the general behaviour observed has been the same in each nominally identical testing case.

4.2.3 Benchmark Tests

Throughout this section, the ambient temperature response of the reinforced concrete columns tested has been presented. The response of the columns is, in general, as expected and no further discussion is needed with regard to their response. This has therefore provided a benchmark against which to compare the response of the remaining heated columns when they are tested residually upon cooling back to ambient temperature. The results of these comparisons are described in more detail in Chapter 5 which is dedicated to the residual performance of the columns.

The remainder of this chapter deals with the response of the columns during the heating and cooling phases of the experiments. Detailing the effect of the individual parameters is discussed in isolation.

4.3 Temperature Evolution Through Columns

Section 3.3.3 presented the experimental setup regarding the radiant panels and their heat flux characterisation/calibration; this section also illustrated an incident radiant heat flux map measured with distance from the radiant panels surface. It is, however, important to note that it is the distribution of internal temperatures within the concrete columns that determines their response when heated, rather than the external heat flux, per se. In order to measure the internal temperatures within the columns, as discussed in Section 3.2.3, thermocouples were cast within the concrete columns during their construction to measure the temperature gradients through their cross sections. Figure 4.3 shows the recorded internal temperatures within the concrete cross-section at mid-height for two identical repeat tests performed with one-sided heating at a peak initial incident radiant heat flux of 70 kW/m^2 (i.e. specimens 30-L60-E15-HF70-T-HL33-(1) and 30-L60-E15-HF70-T-HL33-(2)).

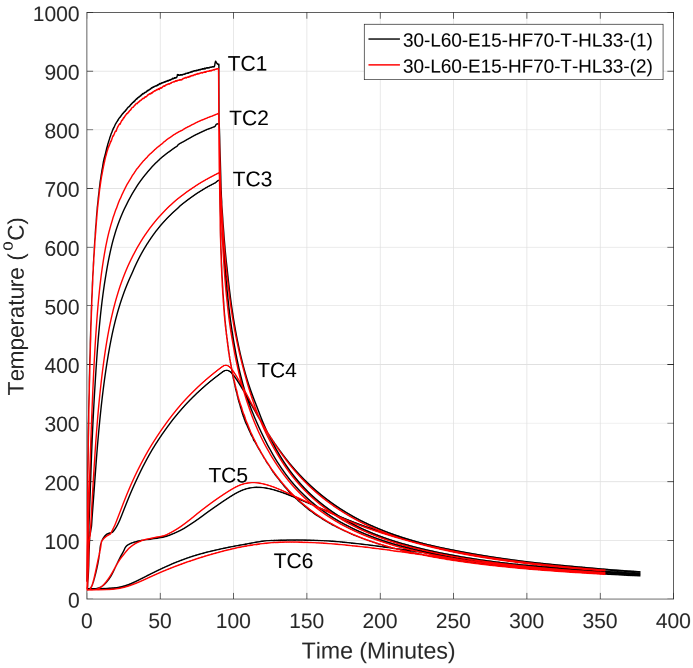


Figure 4.3: Indicative temperature distribution within columns for two specimens subjected to a $70kW/m^2$ heat flux

Figure 4.3 shows the evolution of internal temperatures through two notionally identical columns for the high heat flux (70kW/m^2) exposure used in the experiments. As the heat flux was controlled using the same propane fired radiant panels for each test, and was re-calibrated once every 10 tests, the attained temperature evolution within the columns is consistent between tests.

Figure 4.3 does not however detail tests conducted on reinforced concrete columns that deflected in opposing directions i.e. one column toward the heat source and the other away from the heat source. When loaded and heated the columns deflect, as the radiant panel remains stationary, the heated surface of the column will either (1) get closer to the radiant panel, thus increasing the incident heat flux or (2) get further away from the radiant panel, thus reducing the incident heat flux. This therefore resulted in slight inconsistencies in the temperature evolution of the columns in tests that deflected in opposite directions.

Figure 4.4 shows two experiments conducted on reinforced concrete columns that deflected in opposite directions (i.e. specimens 30-L60-E25-HF70-T-HL33 and 30-L60-E25-HF70-C-HL33).

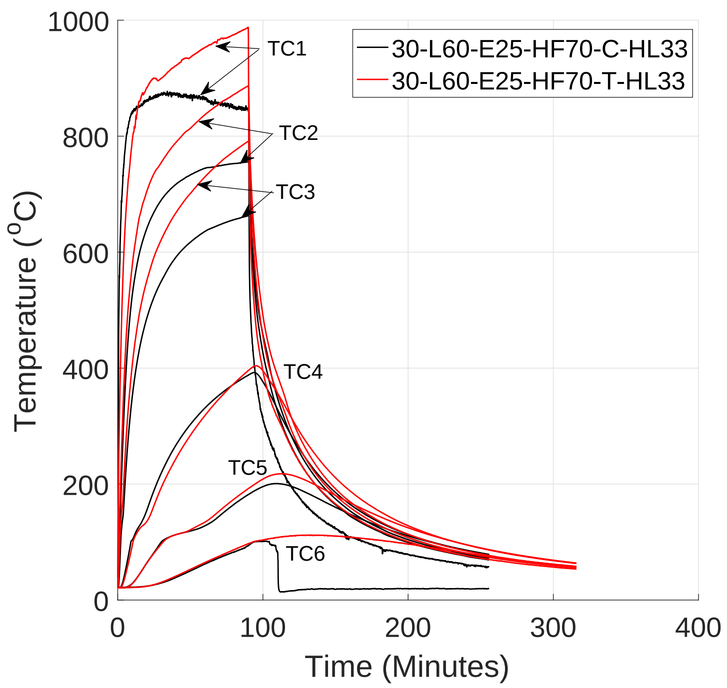


Figure 4.4: Indicative temperature distribution within columns for two specimens exposed to $70kW/m^2$ deflecting in opposite directions

It is clear from Figure 4.4 that the column that deflected toward the heat source achieved higher temperatures throughout the section. The difference in the temperatures amounts to a maximum of 150^{o}C, or around 15% of the total increase in temperature above ambient temperature conditions. Despite this difference, this change in temperature resulted in only slight differences in the structural behaviour due to the fact that the inconsistency occurs at the surface of the column where temperatures are at their highest (and when the concrete is well past the point of being structurally viable). Deeper into the section, where the concrete still contributes to the strength of the section, it is clear from Figure 4.4 that the temperatures become more consistent between tests. This difference in temperatures does not compromise the validity of the conclusions made throughout this thesis because the difference in temperature can be quantified for each of the experiments and can therefore be accounted for in any models produced during a validation. As the difference is at its greatest at the surface and becomes more consistent deeper into the section, the general trend of the behaviour will also be the same, allowing for a qualitative comparison between experiments.

4.4 Typical Response Mechanisms Observed

Throughout the remainder of this chapter, a discussion of the 46 experiments conducted has been discussed. The behaviour of each of the experiments is presented individually to give the reader an idea of the influence of each of the variables on the response of the concrete columns. What is clear from this discussion however, is the fact that the same mechanisms are observed in almost all of the experiments (to differing degrees). Therefore, to provide some context for the response of the columns before delving into the specifics of each experiment, the three main mechanisms that result in increased or decreased deflections in the specimens has been presented.

Thermal Expansion

The first mechanism observed throughout this experimental test series is that of thermal expansion. Thermal expansion occurs when there is an increase in temperature of the concrete columns. As the concrete columns are heated on one side only, there is a temperature profile through the section i.e. one side of the column is hot and the other side is cold. Structurally speaking, this will result in the column bowing in one direction because the degree of thermal expansion will be greater on one side of the column than the other. This response has been detailed in Figures 4.5 and 4.6 where the expected midspan deflection of the column has been plotted indicatively for a “compression heated” and a “tension heated” column.

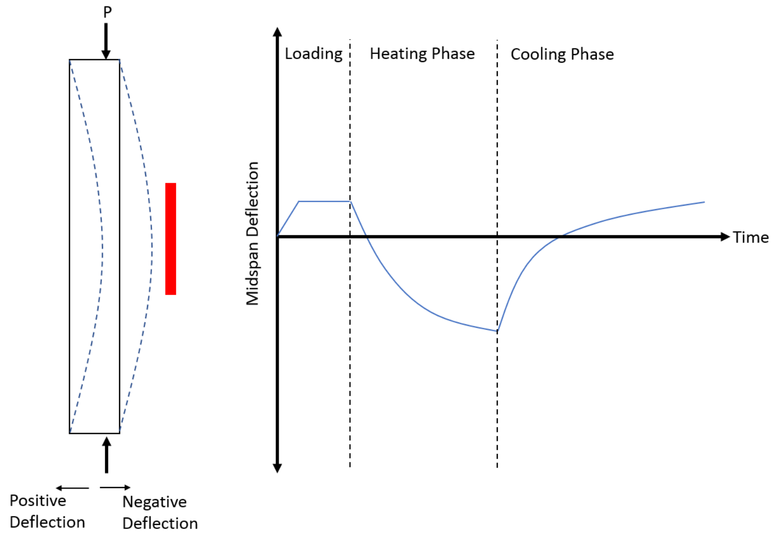


Figure 4.5: Indicative deflections caused only by thermal expansion on a “compression heated” column

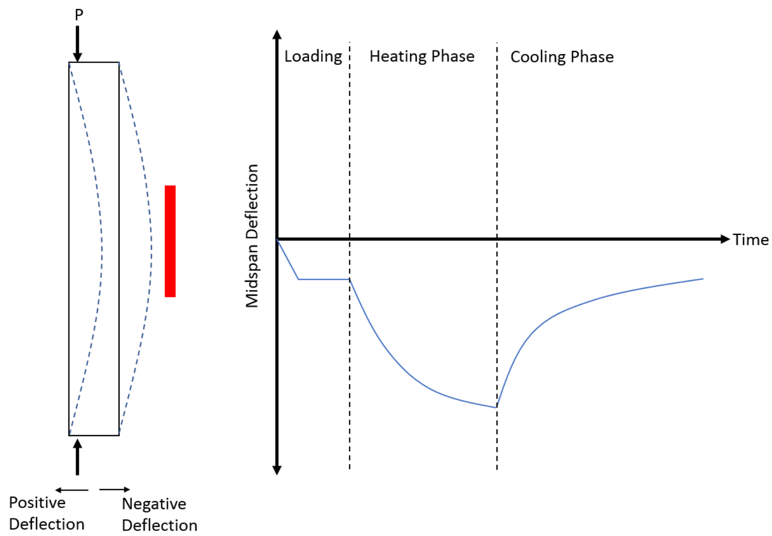


Figure 4.6: Indicative deflections caused only by thermal expansion on a “tension heated” column

As is illustrated in Figures 4.6 and 4.6, it would be expected that the concrete the heated surface would expand to a greater extent than that of the concrete on the unheated surface. This would result in the column deflecting towards the radiant panel during the heating phase in both cases before subsequently recovering in the cooling phase.

Degradation of Concrete at Elevated Temperature

As discussed in Chapter 2, the elastic modulus of concrete decreases with increasing temperature. In the case of this experimental test series the decrease in the elastic modulus of the concrete on one side of the section will result in a reduction in the stiffness of the section. The indicative midspan deflections of the specimens purely as a result of this are detailed in Figures 4.7 and 4.8

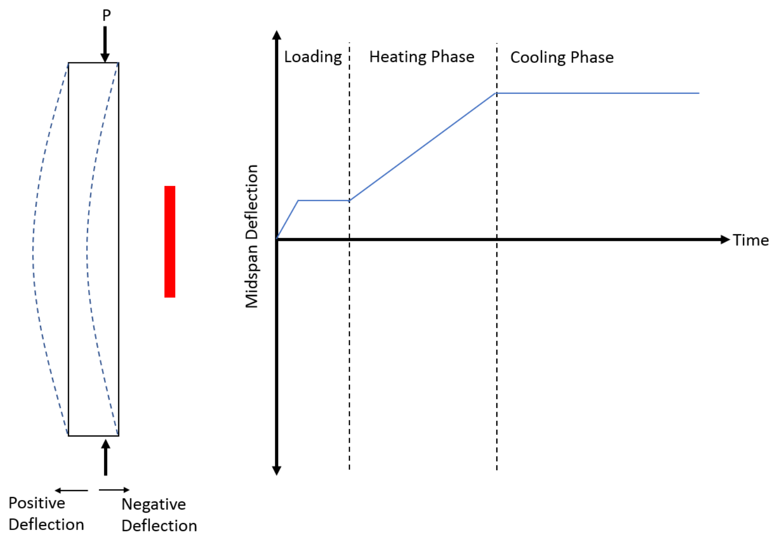


Figure 4.7: Indicative deflections caused only by reduction in the elastic modulus of the concrete on a "compression heated" column

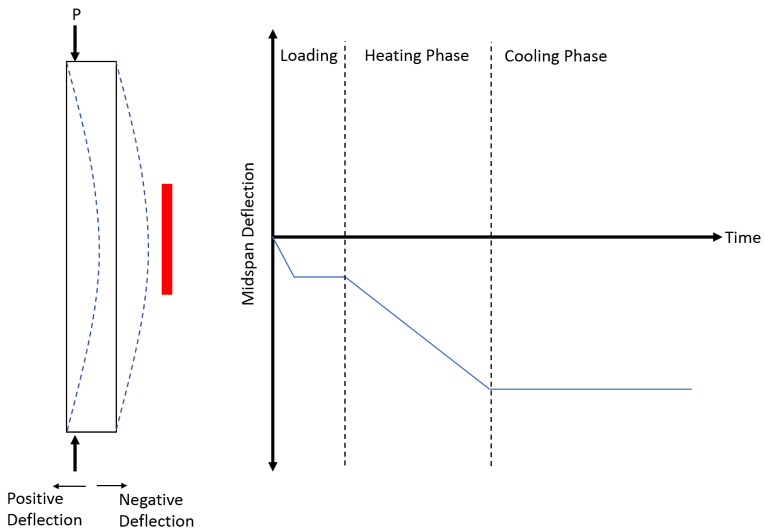


Figure 4.8: Indicative deflections caused only by a reduction in the elastic modulus of the concrete on a "tension heated" column

Purely as a result in the degradation of the concrete at elevated temperatures it would be expected that the reduction in the stiffness of the specimens would result in an increase in the initial deflections of the columns. This is with the caveat that columns heated on the "tension face" with a very small eccentricity may see a reversal in the direction of the deflections as a result of the weakening of the concrete on one side of the column resulting in the "tension eccentricity" becoming a "compression eccentricity". As concrete does not degrade further in the cooling phase, it would be expected that there would be no change in the deflections after the heat source is removed. There has been some research discussing additional loss of material properties of concrete in cooling. BS EN 1994-1-2 suggests an additional 10% loss in strength after cooling to ambient temperature conditions. If this is also true for the elastic modulus of the concrete, then perhaps an additional small increase in the deflections would be observed.

Transient Thermal Creep

Transient thermal creep, as discussed in Chapter 2 is a phenomenon that results in non-recoverable deflections of concrete that is loaded and then heated. In the context of this experimental test series, this can result in a number of very interesting behaviours. Similar to the other mechanisms discussed, the indicative deflections caused by transient thermal creep are illustrated in Figures 4.9 and 4.10.

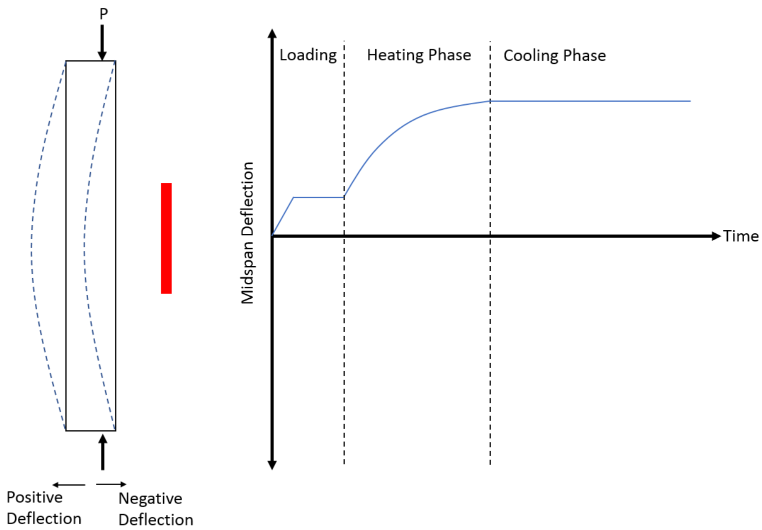


Figure 4.9: Indicative deflections caused only by transient thermal creep on a "compression heated" column

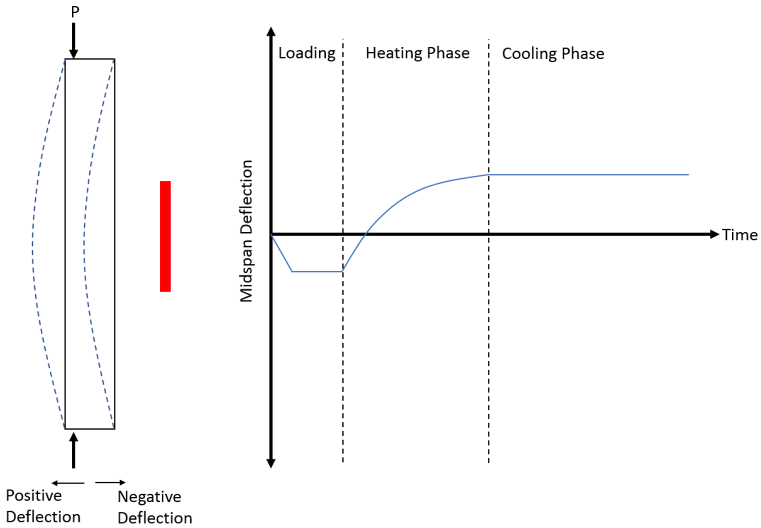


Figure 4.10: Indicative deflections caused only by transient thermal creep on a "tension heated" column

Transient thermal creep results in additional contraction of compressed concrete as the temperature of the concrete rises. This means that the heated concrete on one side of the columns tested would contract as a result. Therefore, if the columns were to deflect purely as a results of transient thermal creep, it would be expected that the columns would begin to bow away from the heat source during the heating phase. As this phenomenon results in non-recoverable contractions of the concrete, it would be expected that nothing would happen in the cooling phase after the heat source has been removed.

The three mechanisms described in this section; thermal expansion, degradation of the concrete, and transient thermal creep, are the three main mechanisms that result in the deflections discussed in the following sections. These mechanisms contribute to differing degrees under different loading and heating conditions which results in a number of the unusual deflection plots discussed. However these mechanisms, in there simplest form, are the mechanisms that contribute to all of the behaviour discussed herein, and will be referred to when discussing the experiments conducted.

4.5 Concrete Strength

4.5.1 Significance of Concrete Strength

The only parameter investigated within the experimental program that has an effect on the material construction of the specimens was the strength of the concrete used. The remaining parameters are based upon differing the loading and heating scenarios. 24 concrete columns were tested using 30MPa concrete and 22 concrete columns were tested using 50MPa concrete. It is therefore not only the strength of the concrete that differs between the two batches, the mix design differs in that super-plasticiser has been added to the stronger mix to increase its strength. This could in turn not only change the strength and stiffness of the material, but also its thermal properties and make it more prone to spalling during exposure to elevated temperatures [39]. The propensity of concrete to spall and the design of individual concrete mixes has not been discussed in detail as part of this thesis and is outside the scope of this experimental test program (no spalling was observed in any experiment conducted as part of this experimental test series). Instead, it has been recognised that each individual mix design has its own material properties and characteristics and any conclusions and qualitative comparisons are discussed with these differences in mind. There are general structural configurations, trends and material behaviours that hold true for all mix designs, it is these behaviours that is compared and contrasted to investigate how they might interact with one another under different loading and heating conditions.

As can be expected from such a significant difference in the strength of the material used, the response of the two types of columns varied greatly. This alone is of no great significance. Basic mechanics would show that a stronger, stiffer column of the same dimensions will hold more load and deflect less relative to the weaker column. What is however interesting, is the response of the two types of column when they are subjected to similar heating scenarios. These responses are discussed in greater detail throughout this section and built upon further throughout Chapter 4.

One limitation of the experiments conducted for this thesis however, is the fact that columns were subjected to a load that was a function of the total capacity of the column under that particular load eccentricity i.e. 60% of the columns' ambient temperature capacity. This results in columns of different strengths being stressed to different levels for each of the scenarios. It is therefore difficult to draw direct comparisons between the loaded tests for the 30MPa and 50MPa concrete. It is, however, possible to compare the response of the columns under small loads. Each load eccentricity and heating condition was also tested under a minimal load of 10kN for both types of concrete to investigate the effect of the applied load during heating on the residual response of the columns. As the concrete has been stressed and heated to the same degree for these tests, it is possible to compare and contrast their responses directly.

4.5.2 Concrete Strength and Low Load

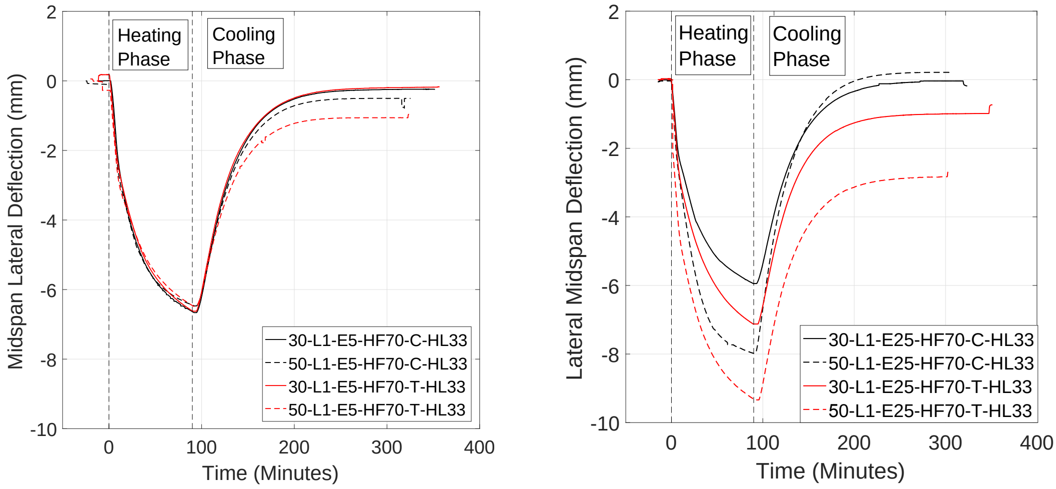
The 30MPa and 50MPa columns were tested under a minimal load of 10kN to determine the difference in the deflection history of the columns under identical heating conditions for different strengths of concrete. Four conditions have been tested for each concrete strength, these conditions are detailed in Table 4.2.

Table 4.2: Low Load Tests Conducted

Concrete Strength (MPa)	Load (kN)	Load eccentricity (mm)	Heat Flux (kW/m^2)	Side Heated
30	10	5	70	Tension Face
	10	5	70	Compression Face
	10	25	70	Tension Face
	10	25	70	Compression Face
50	10	5	70	Tension Face
	10	5	70	Compression Face
	10	25	70	Tension Face
	10	25	70	Compression Face

In conducting these experiments, the effect of loading on the columns is minimised. It would therefore be expected that the columns would deflect purely as a function of thermal expansion. Based upon the current design guidance within BS EN 1992-1-1 [4] these deflections and strains would then be expected to fully recover upon cooling back to ambient temperature without, for example, transient thermal creep taking effect. A load of 10kN was selected as it was the minimum load that could be reliably maintained by the hydraulic pump used to feed the loading jack. This amounted to around 1-3% of the total capacity of the strong and weak columns respectively.

The following figures compare the midspan deflections of the columns subjected to an identical heat flux for 90 minutes.



(a) 5mm load eccentricity　　**(b)** 25mm load eccentricity

Figure 4.11: Midspan deflection of low load columns tests at (a) 5mm and (b) 25mm load eccentricities

As can be seen from Figure 4.11a, all four columns tested under low load conditions at a 5mm eccentricity experienced almost identical deflection responses during the heating and cooling phases of the experiments. The 10kN load was applied at -20 minutes, small deflections are seen at this point due to the fact that the load applied is low compared to the total capacity of the columns. At 0 minutes, the columns were exposed to a $70kW/m^2$ heat flux for a total of 90 minutes. Almost immediately upon being exposed to the heat flux, the differential thermal expansion through the depth of the columns results in lateral deflections and bowing of the columns. This thermal bowing increases to a total of roughly 6.5mm after 90 minutes of heating, after which the heaters are switched off. The columns are then left for several hours to return to ambient temperature conditions, at which point it can be seen that the column deflections recover almost entirely to the condition they were in before being heated.

As the load is applied at a small eccentricity in Figure 4.11a, this would be expected considering that the pivot point for the columns is more or less in the centre of the column. There is therefore no significant difference between the "tension heated" and "compression heated" tests as they are pivoting roughly around the same location in the column. The difference between the pivot point for both sets of tests is 10mm (6.6% of the width of the column). In addition to this, it is also clear that there is no great significance between the deflection history of the columns of different concrete strength. Considering that the columns were subjected to an identical heat flux in the same location for the same period of time, this would suggest that the thermal properties of the two types of concrete are similar under unloaded conditions.

The results of the 5mm eccentricity tests are however in contrast to those for the columns subjected to the same heating and loading conditions under a 25mm load eccentricity, as illustrated in Figure 4.11b. It appears that the "tension heated" tests deflect more than the "compression heated" tests, and do not recover to the pre-heated values after cooling. It should be noted that, despite being subjected to the same load, the moment in the sections is now five times that of the 5mm eccentricity tests in Figure 4.11a, due to the fact that the load eccentricity (25mm) is five times greater. Taking thermal expansion into account, and looking at the "tension heated" tests, the expansion of the surface of the columns upon heating would work alongside the moment induced in the section to increase the deflection in the column. This increase in the deflection would then induce secondary moments, increasing the moment in the section even farther. In comparison to the "compression heated" tests where any deflections caused by thermal expansion of the concrete would decrease the moment in the section resulting in smaller deflections compared to the "tension heated" tests.

In addition to this, it is also clear from Figure 4.11b that the columns consisting of higher strength concrete experience higher deflections than those of the lower strength concrete. As the stronger concrete columns are stiffer than the weaker columns and similar temperature gradients were observed through all of the columns, this would suggest that the increase in deflections is due to increased thermal expansion of the stronger concrete columns. This observation cannot however be verified due to the fact that the coefficient of thermal expansion of the concrete has not been directly measured.

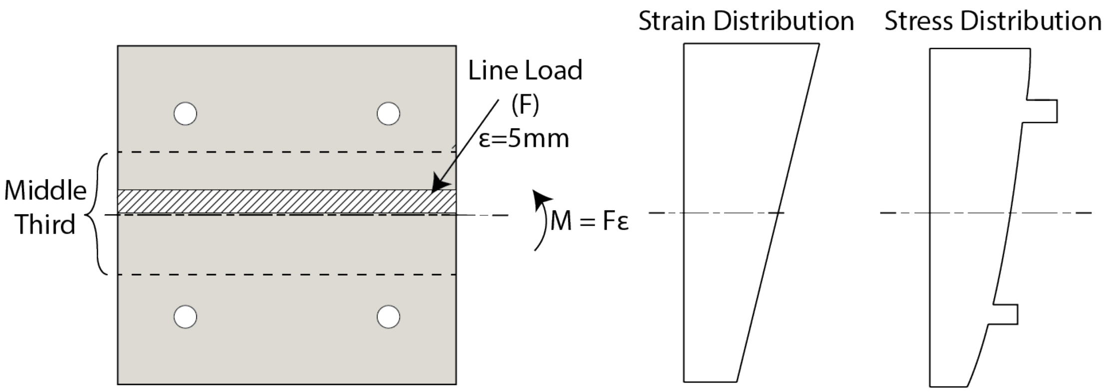
For the experiments conducted on reinforced concrete columns subjected to a very small sustained axial load, as compared to their ambient temperature capacity, differing the conditions by changing the side heated or the load eccentricity has no great effect on the response of the columns. All of the columns experience lateral deflections in the same direction (towards the heat source) which are all at least partially recovered upon cooling. This is in part due to the fact that, under small axial loads, concrete is expected to experience some thermal expansion which is then recovered upon cooling. This has been shown to be true in Figure 4.11 where differential thermal expansion through the depth of the section has caused thermal bowing which has then been recovered during the cooling phase of the experiment. Most of the discussion from now will concentrate on the comparison of these tests to the concrete columns loaded to 60% of their ambient temperature capacity, where significantly different behaviour have been observed.

4.6 Load Eccentricity

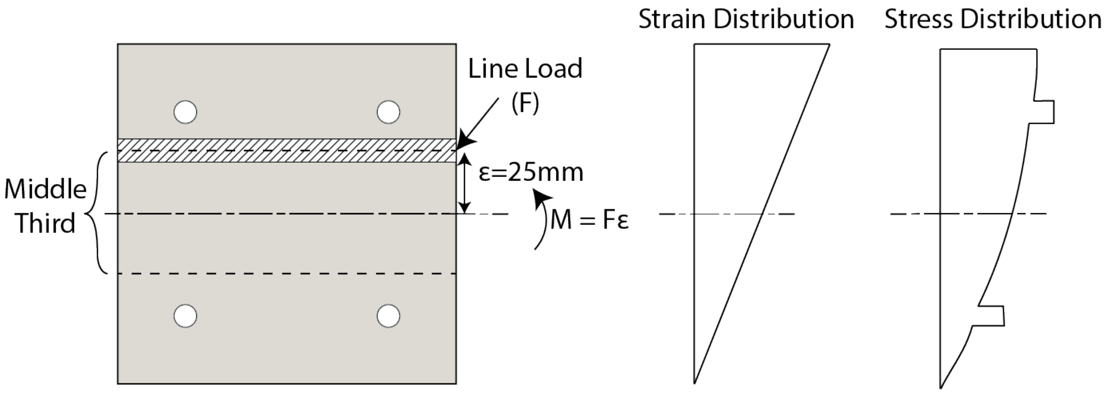
4.6.1 Significance of Load Eccentricity

Load eccentricity is an important factor in the response of a compressive structural element. It will change the location at which the load is applied, in turn changing the moment applied to the column as a result of the secondary moments induced. This moment will then increase or decrease based upon whether the deflection of the column is toward or away from the load point respectively. Under certain conditions, to be discussed in Section 4.6.5, this, in combination with the deflection of the column, may result in a positive moment becoming a negative moment under certain scenarios and deflections (or vice-versa). The second significant point to make is that changing the location of the load eccentricity will change the pivot point in the columns, altering the impact of thermal expansion and contraction on the deflections.

Looking specifically at the stress distribution within the concrete section when subject to different load eccentricities, at the start of the test, every column will be subject to only compressive stresses. This is due to the fact that both eccentricities (5mm and 25mm) are within the middle third (kern) of the section. As a result, the moment induced within the section is not great enough to overcome the compressive stress induced by the axial force at the extreme fibres. However, upon heating, lateral deflections will result in secondary moments which can either increase or decrease the total moment in the section. Therefore, after being exposed to elevated temperatures, the reinforced concrete columns may well be subject to compressive forces on one face and tensile forces on the other. This is is important when considering what may happen with regard to thermal expansion or transient thermal creep throughout the test. Figure 4.12 shows the stress distribution within the reinforced concrete columns for each load eccentricity at the start of the experiment before being exposed to elevated temperatures.



(a) 5mm load eccentricity



(b) 25mm load eccentricity

Figure 4.12: Indicative stress-strain distributions for columns subject to differing load eccentricities (secondary moments have been ignored)

As detailed in Figure 4.12, due to the fact that the load is applied within the middle third of the section, the section only experiences compression stresses. However, for clarity within this thesis, the face under the highest level of compression has been referred to as the "compression face", and the face under the lowest level of compression has been referred to as the "tension face".

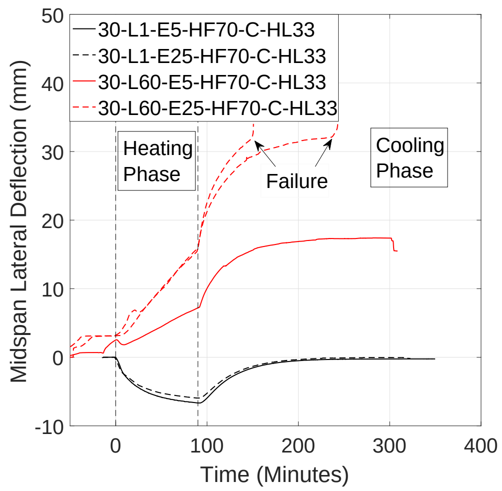
4.6.2 Eccentricity and Concrete Strength

For a full discussion on the relationship between the above two parameters, See Section 4.5.2. This discussion is based purely around the columns tested at low load levels due to the fact that, in the low load condition, they are subjected to the same 10kN axial force. However the tests subjected to higher loads are subject to differing axial forces, therefore making direct comparison of concrete strength and eccentricity is a difficult task within this experimental program conducted.

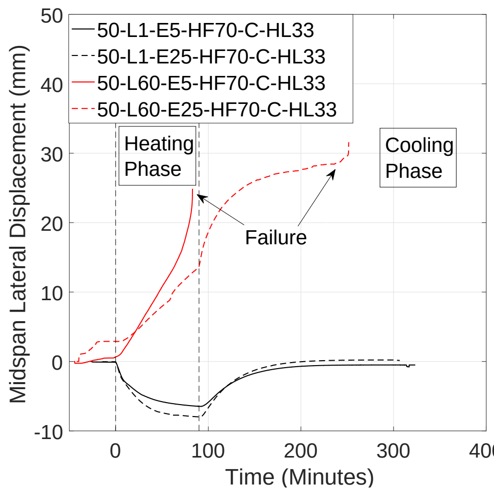
4.6.3 Eccentricity and Magnitude of Load

Much of the response of the concrete columns tested under low load conditions is heavily influenced by both the concrete mix and the load eccentricity. As a result, most of the discussion regarding the effect of load eccentricity on the unloaded specimens has been provided in Section 4.5.2. Section 4.5.2 concluded that the load eccentricity had an effect on the columns under the low load condition because it changed the pivot point of the columns, therefore magnifying or reducing the influence of thermal expansion on the thermal bowing of the columns. In addition to changing the pivot point, the change of eccentricity increased or decreased the moment within the section, even for the relatively small axial force applied. This resulted in secondary moments having a greater influence on the deflections by either benefiting the column (in the “compression heated” case), or acting to hinder the response of the columns (in the “tension heated” case).

In the case of higher load levels, the increase in axial load will also increase the moment within the column, therefore significantly altering the response of the columns when exposed to elevated temperatures. Figure 4.13 details the significantly varying deflection history of columns subjected to differing load levels while being heated on the compression face.



(a) C30 concrete columns



(b) C50 concrete columns

Figure 4.13: Midspan deflection of compression heated tests for both (a) C30 and (b) C50 concrete columns

It is clear from both Figure 4.13a and Figure 4.13b that the increase in load results in significantly different structural responses by the reinforced concrete columns. The unloaded tests for both concrete mixes tested respond in a similar way in that, due to the small load imposed on the columns, there are negligible deflections as a result of the mechanical load at the start of the test. When heated, the low load tests exhibit thermal bowing toward the heat source (negative deflection) as a result of the temperature gradient through the columns and, after a 90 minute period of heating, the columns cool back to ambient temperature at which point there is almost no residual displacement in the elements.

This is in contrast to the tests conducted on concrete columns loaded to 60% of there ambient temperature capacity. At the start of the tests, as a result of the increased load, the columns bow away from the heat source (positive deflection), as expected. This load is then maintained for a period of 20 minutes to allow the column to settle until the heat source is switched on at 0 minutes. It is at this point the response of the columns are rather counter-intuitive. It would be expected upon exposure to elevated temperatures on one side that the column would bow toward the heat source as is the case for the low load tests in Figure 4.13a and Figure 4.13b. The differential expansion of the concrete section on the side of the heat source would be expected to result in deflections which bring the column closer to the heat source (negative deflection in Figure 4.13). This however, is not the case. Upon exposure to elevated temperatures on the "compression face", the columns continue to deflect away from the heat source. After 90 minutes of heating the heat source is removed and the columns are allowed to cool down to ambient temperature conditions. It can be seen that, upon the onset of cooling, the columns begin to rapidly deflect away from the heat source for the remainder of the experiment, until either the column fails or a new state of equilibrium is achieved when the column has returned to ambient temperature.

Heating Phase

As previously noted, upon being exposed to elevated temperature, the loaded columns deflect away from the heat source. This may be explained by the behaviour of concrete when it is exposed to elevated temperatures while mechanically stressed. As discussed in Chapter 2, transient thermal creep occurs within concrete when it is stressed to a certain level while also being exposed to elevated temperatures, therefore contracting as opposed to exhibiting thermal expansion. In this particular case, when the columns are resisting both an axial force and bending moment, the stress distribution varies over the cross-section. Figure 4.14 illustrates what the stess and strain distributions within the section would indicatively look like under these conditions.

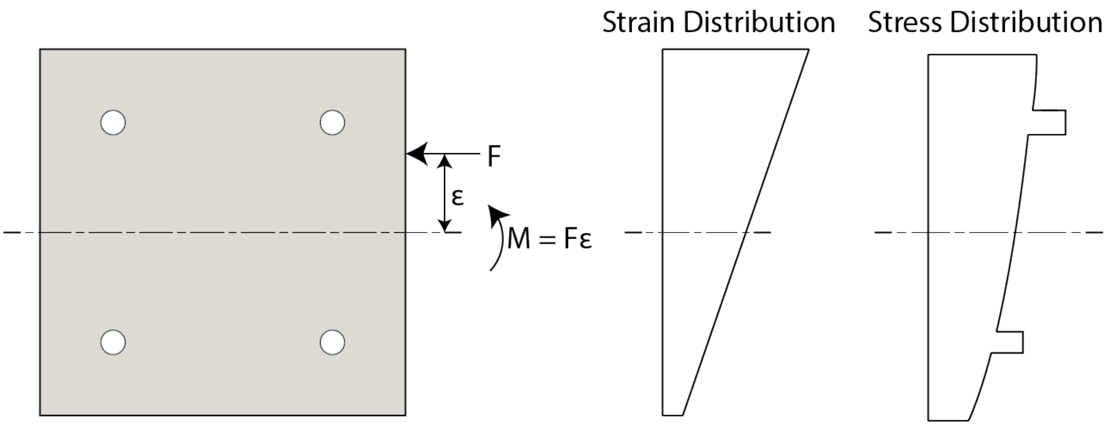


Figure 4.14: Indicative stress and strain distribution within columns

Figure 4.14 shows that the stress is at a maximum at the extreme fibre on the side where the eccentric force was applied. The stress at this point is greater than the threshold required for transient thermal creep to take place. Therefore, when exposed to heat on the "compression face", the concrete at the extreme fibre contracts and results in bowing away from the heat source, as observed in Figure 4.13a and Figure 4.13b.

In addition to transient thermal creep resulting in the column bowing away from the heat source, the increase in temperature within the section will result in a weakening and softening of the concrete. Due to the thermal gradient in the section resulting from the localised heating of the column, the column will be weaker and less stiff on the side exposed to the heat source. This will move the centre of force of the section away from the heated surface of the column which, in turn, increases the moment within the section. Therefore increasing deflections even more. This process of the centre of force moving as a result of the increased temperatures has been detailed in Figure 4.15. For a full explanation of this phenomenon, see Section 6.3, where it is discussed as part of the procedure for analysing the tests using a sectional analysis model.

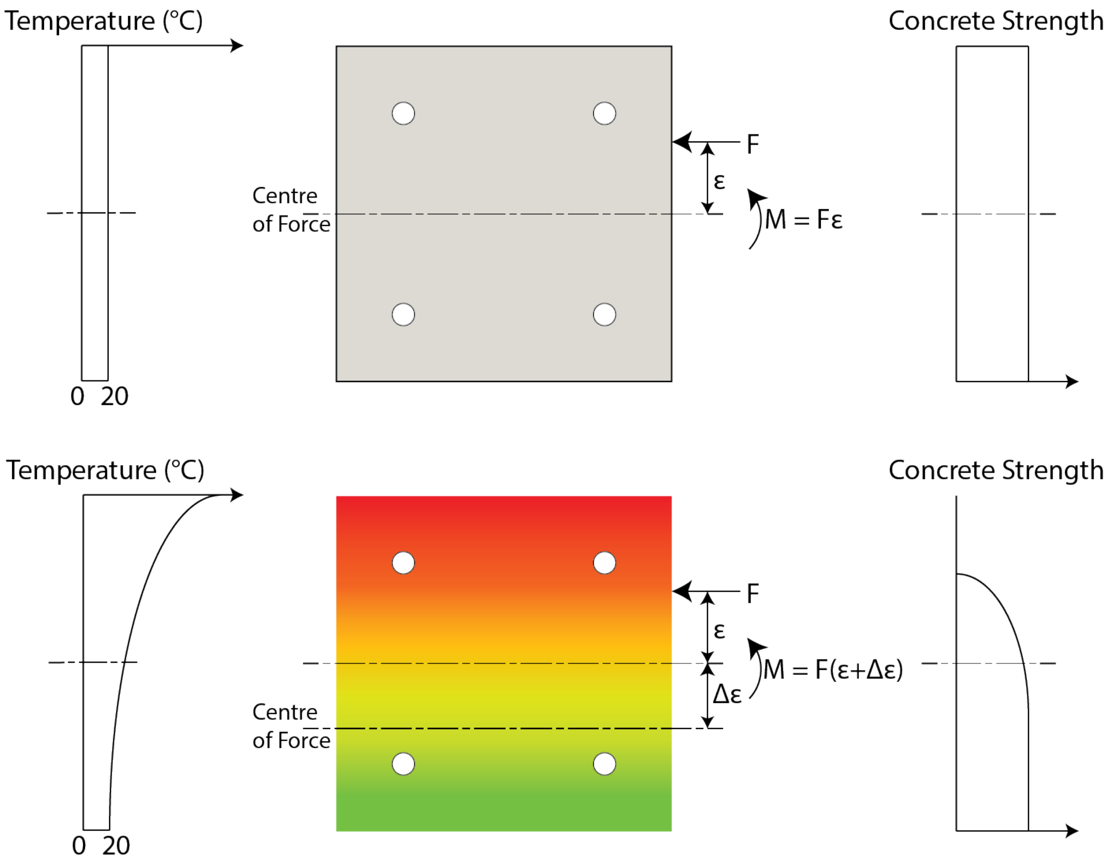


Figure 4.15: Increase in bending moment as a result of temperature gradient through section

As a result of transient thermal creep within the concrete on the heated surface and the weakening of the concrete, resulting in increased moment in the section, the bowing of the column away from the heat source can be qualitatively explained. In addition to this, as with all columns, bowing will result in secondary moments within the section, further exacerbating the deflections.

Cooling Phase

Figure 4.13a and Figure 4.13b show that both the high load and the low load configurations deflect away from the heat source at the onset of cooling. The unloaded columns for both the strong and weak columns recover almost entirely back to zero deflections in the column. This behaviour is expected at this point owing to the fact that the heat source is switched off and the surface of the concrete immediately cools rapidly (by roughly 550^{o}C in 10 seconds). The thermal wave within the concrete is however continuing to move through the section and much of the concrete is still to achieve its maximum temperature. As a result of this rapid cooling at the surface of the column, the extreme fibre of the column will contract, worsening the bowing in the column. As the column cools, this contraction will travel through the column, increasing both the deflections in the columns and the secondary moment as a result. This continues until the column has returned to ambient temperature conditions roughly 4 hours after the heat source is switched off in most cases.

For the lone column that survived until the end of the test in Figure 4.13a (C30 concrete subjected to 60% load capacity at 5mm eccentricity, 30-L60-E5-HF70-C-HL33), the load was removed after it had cooled back to ambient temperature. It would classically be expected that, after cooling back to ambient temperature and having the load removed, the deflections in the column should return to zero. It is, however, clear from Figure 4.13a that this is not the case. When the load is removed, the central deflection in the column moves from 17.38mm to 15.59mm from its original zero position before the test, a total recovery of only 10% of the deflection in the column. This may again be a result of transient thermal creep within the column during the heating phase which results in non-recoverable strains within the concrete. If this phenomenon affects one side of the column more than the other, it will result in a permanent bowed shape of the column when cooled back to ambient temperature. This in turn means a permanent increase in the eccentricity of the load and moment due to the bowing of the column. This should be considered when engineers undertake post-fire condition assessments of fire-damaged concrete structures.

Failure Mechanisms

In Figure 4.13a and Figure 4.13b most of the columns loaded to 60% of their ambient temperature capacity failed before the end of the test. This occurred due to the fact that, despite the axial load remaining constant, the deflection of the columns results in secondary moments being induced. As deflections increase, so does the secondary moment, resulting in failure of the column when the secondary moment becomes sufficiently large. For a full discussion of secondary moment within the columns and a comparison with the model developed, see Chapter 6.

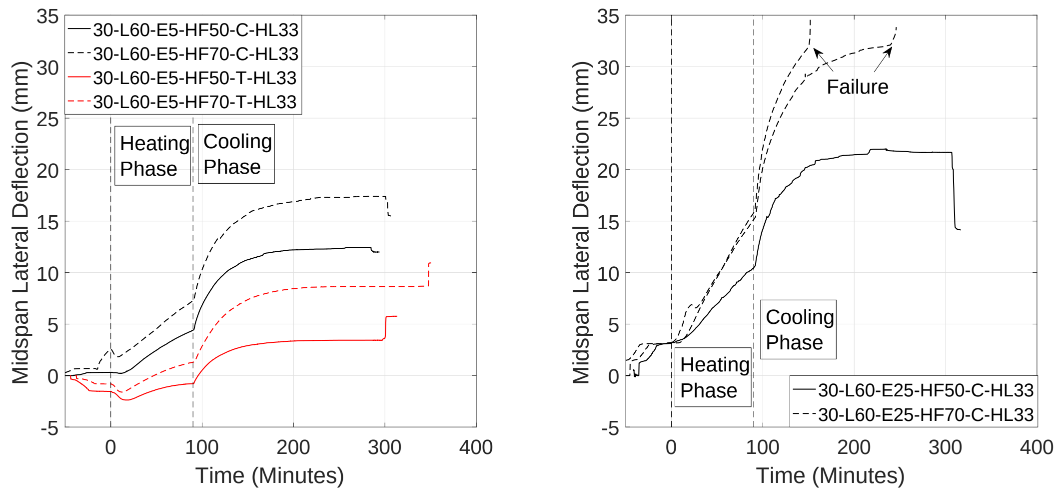
In Figure 4.13a and Figure 4.13b, four of the columns failed during the test. One during heating and three during cooling. The failure mechanism for all of the columns was identical to that of the failure mechanisms at ambient temperature illustrated in Figure 4.2. A failure in bending was observed in all cases and, unlike the ambient temperature tests, the location of the failure was identical for all columns. This can be attributed to the fact that the columns were all heated locally and damaged in the same location. Figure 4.16 displays the typical failure mechanism for the four columns that failed during the tests in Figure 4.13a and Figure 4.13b.

Figure 4.16: Typical mode of reinforced concrete columns in heating and cooling

It is important to note that the current design guidance allows for the columns tested to be analysed up until the end of heating. Looking at Figure 4.13a and Figure 4.13b it is clear that this is only half of the story when it comes to the damage sustained by the column as a result of exposure to elevated temperatures. In this particular arrangement the central deflections of the column double after the heat source has been switched off (i.e. the fire has been put out). This has important repercussions for fire fighters and those entering a building to assess damage as a result of fire. For a full discussion of potential consequences and the significance of these findings in "real life" scenarios, see Section 4.11 where theoretical examples have been provided to discuss and challenge the current thinking in design and construction of “fire safe” buildings in terms of concrete columns’ performance during and after fire.

4.6.4 Eccentricity and Heat Flux

When changing the eccentricity of the load in combination with the heat flux applied, the expected conclusion would be that, the larger the heat flux, the larger the displacements and the strains in the column due to the increased damage caused by the higher temperatures. This is because, as the heat flux increases, the damage penetrates deeper into the column and, as discussed in Section 4.6.3, the increased damage results in both a weakening of the column as well as increased moments within the column. Therefore increasing the severity of the deflections and “damage” to the columns. This has been illustrated in Figure 4.17a and Figure 4.17 where it can be clearly seen that increasing the heat flux on the columns increases the severity of the deflections.



(a) 5mm load eccentricity

(b) 25mm load eccentricity

Figure 4.17: Midspan deflection of tests at differing heat fluxes on 30MPa concrete columns for (a) 5mm and (b) 25mm load eccentricity

At first glance it appears that all of the columns behaved in a similar fashion in that they all deflected away from the heat source during heating, this then worsened during the cooling phase for all of the columns in both Figure 4.17a and Figure 4.17b. It is, however, also clear from Figure 4.17a that an unexpected behaviour took place at around 20 minutes for the "tension heated" specimens. For these experiments, where the column was subjected to a 5mm load eccentricity and a 70kW/m^2 heat flux on the tension face, the column began to bow in one direction (towards the heater) and then, after roughly 15-20 minutes of heating, these deflections began the reverse and the column recovered. Beyond this point, the columns recover fully back to zero and transitioned to bending in the other direction, away from the heat source. This is in contrast to the "compression heated" columns that deflected away from the heater during both the heating and cooling phases as described in Section 4.6.3.

Compression Heated Columns

What is clear from Figure 4.17a and Figure 4.17b is that the “compression heated” columns behaved in a similar manner to those is Figure 4.13 in that, from the moment of heating, the columns deflected away from the heat source. This was then exacerbated in the cooling phase during which the deflections sustained doubled. For a full discussion on this behaviour and discussion on the mechanisms behind it, see Section 4.6.3. What can, however, be concluded from Figure 4.17 is that, for the “compression heated” columns, the larger the incident heat flux applied to the column, the greater the damage and resulting deflections it sustains. This is due to the fact that, as discussed in Section 4.6.3, decay of the properties of the concrete and stiffness of the column as well as transient thermal creep are the likely causes of this behaviour. Therefore increasing the heat flux the columns are exposed to will both (1) increase the maximum temperature of the column and, (2) speed up the thermal wave travelling through the section. This will in turn increase the severity of the transient thermal creep in the heating phase, increasing the deflections sustained. In the cooling phase, as the thermal wave will travel deeper into the concrete over the same period of time, more of the concrete in the section will shrink during cooling, increasing deflections beyond the tests conducted with a lower heat flux.

Tension Heated Columns

As previously discussed, the tension heated columns in Figure 4.17a display an interesting deflection history in that they begin to deflect toward the heat source and, after 20 minutes of heating, recover and begin the deflect in the opposite direction.

This behaviour was not foreseen prior to the experiments being conducted and particularly illustrates the need for a mechanics based approach when it comes to fire resistance testing of concrete compression elements. The eccentricity applied to the column transitioned from inducing a positive moment, to inducing a negative moment within the column itself, a behaviour that cannot be captured by the traditional approaches that only explicitly consider reductions in strength and stiffness of the concrete.

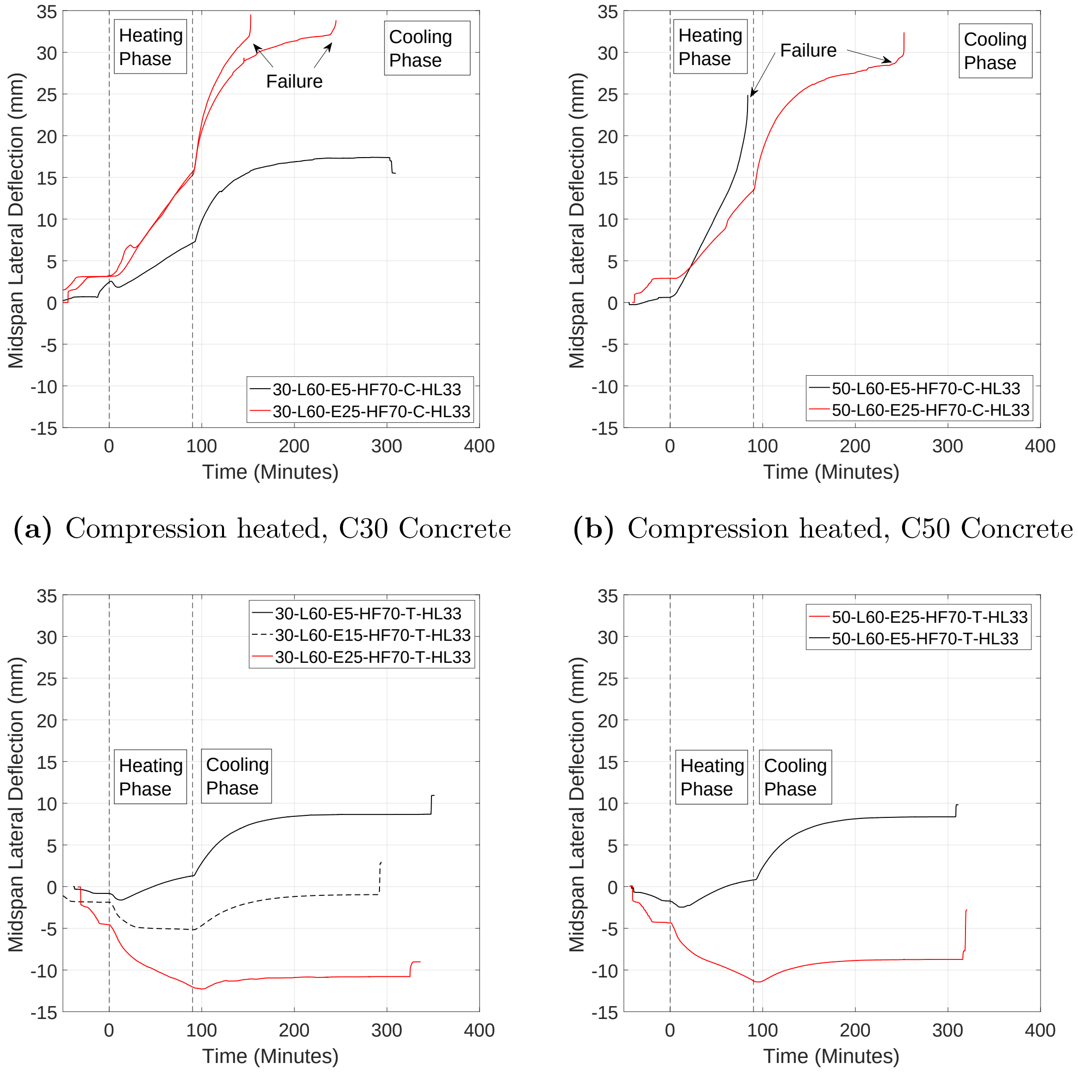
The mechanics of this behaviour can possibly however be understood considering the properties of concrete at elevated temperature. As discussed in Section 4.6.3, the bowing of the column may be explained by transient thermal creep at the extreme fibre of the column where the concrete is stressed to a high level in the "compression heated" scenario. This is not the case for the "tension heated" specimens, the extreme fibre exposed to the heat source is at the lowest stress level. The eccentricity of the load is however only 5mm, well within the middle third of the column, therefore resulting in compression over the entire section as illustrated in Figure 4.12. This is in contrast to the 25mm eccentricity which is on the very edge of the middle third of the column, moving the neutral axis to the edge of the column with no stress at the extreme fibre. Therefore, the "tension face" for the 5mm case will be under a compressive stress, increasing deeper within the section, further away from the heated surface. This has been illustrated previously in Figure 4.12a.

Taking this into consideration it is clear that, despite being subjected to 60% of the load capacity, the columns loaded at a 5mm eccentricity are only stressed to around 10% of there capacity on the heated face of the column, increasing to 30% on the unheated face. It is generally accepted in the current literature that, at a stress of 10% of the capacity of the concrete, there will still be expansion in the concrete [41]. Contraction of stressed concrete at elevated temperatures will begin to occur at around a stress of 20% of the capacity of the concrete [41]. Therefore upon exposure to elevated temperature, the surface of the concrete will expand as a result of the relatively low stress that it is subjected to, resulting in the column deflecting toward the heat source (negative deflections in Figure 4.17). As the thermal wave travels through the section to the concrete stressed to a higher level, transient thermal creep will begin to take effect as with the compression heated specimens This will result in a change in direction of the bowing in the column. Should the test continue for enough time, the column will begin to bow away from the heat source, changing the eccentricity of the load and thus becoming "compression heated" and displaying the same behaviour as "compression heated" columns. This is the behaviour that is illustrated in Figure 4.17a for the "tension heated" samples. This behaviour holds true for all other specimens subjected to load and elevated temperature at a low load eccentricity. Due to the fact that the eccentricity of the load under the "tension heated" scenario resulted in this behaviour, a further level of eccentricity was investigated to find the critical point at which this transition may occur. For details of the tests conducted at a 15mm load eccentricity, see Section 4.6.5.

4.6.5 Eccentricity and Side Heated

When changing the eccentricity of the load in combination with the side heated, the stress distribution within the column will be changing along with the direction of bowing. As a result of changing the side heated, the effects of thermal expansion and transient thermal creep will have different levels of influence in the response and the bowing of the column and, therefore, the secondary moments induced. This has been briefly discussed in Section 4.6.4.

Figure 4.18 displays the central deflections of a number of the experiments conducted, and compares the effects of differing the eccentricity with the side exposed to elevated temperatures.



(a) Compression heated, C30 Concrete **(b)** Compression heated, C50 Concrete

(c) Tension heated, C30 Concrete **(d)** Tension heated, C50 Concrete

Figure 4.18: Midspan deflection of tests at differing load eccentricities for (a) "compression heated", C30 (b) "compression heated", C50 concrete columns (c) "tension heated", C50 concrete columns (d) "tension heated", C50 concrete columns

Figure 4.18 details the results of nine tests conducted on concrete columns at differing eccentricities and with opposing sides heated. It is clear that, as discussed in Section 4.6.4, the tension heated columns behave in a significantly different fashion from that of the ”compression heated” columns. All of the compression heated specimens bend away from the heat source during both heating and cooling, whereas the tension heated specimens tend to bend toward the heat source during heating and away during cooling. This is most likely the result of transient thermal creep of concrete at elevated temperature. For a full discussion on why there is such a pronounced difference in the behaviour of the columns heated on opposing sides, see Section 4.6.4.

It would be expected that, in such a test setup, higher load eccentricities would be more detrimental to the columns. This does not appear to be the case in Figure 4.18b, however, it is important to note that, despite the fact that the eccentricity is the same, the load applied is not. The load applied to the columns was 60% of the columns ambient temperature capacity. Therefore the tests depicted in Figure 4.18 have each been subjected to a different load and moment. Despite not being able to draw direct comparisons between the tests, Figure 4.18 clearly depicts the indicative behaviour of the columns under different load-moment combinations.

One note that is clear from the behaviour of the columns depicted in Figure 4.18 is that a column heated on the higher stressed “compression face” is likely to bow away from the heat source as a result of transient thermal creep. It is however more difficult to determine the behaviour of the columns during the heating phase when exposed to elevated temperatures on the lower stressed “tension face”.

Behaviour of the “tension heated” Columns

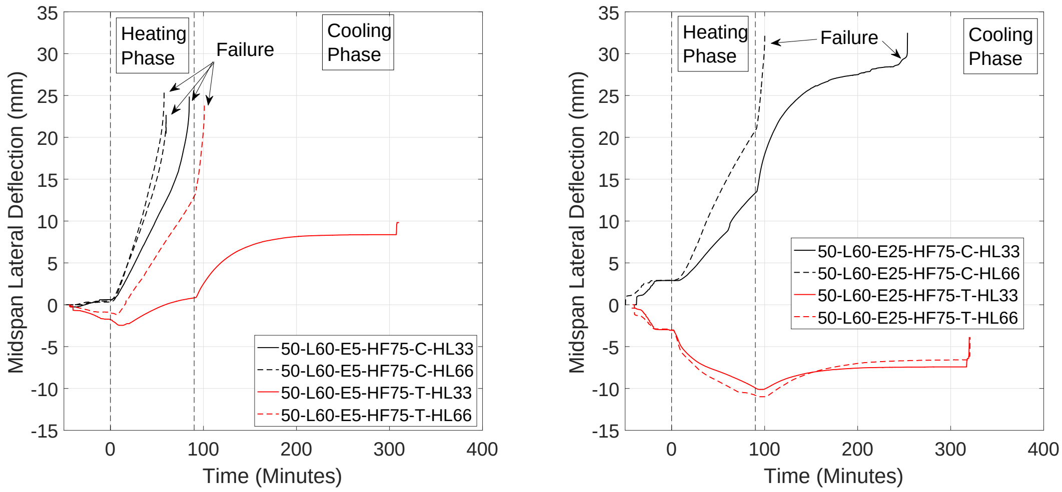
As is clear from Figure 4.18c, an additional value of load eccentricity, 15mm, was investigated. As can be seen in both Figure 4.18c and Figure 4.18d, the columns subjected to a 25mm load eccentricity bend toward the heat source due to thermal expansion of the extreme fibre exposed to the heat source. However, the columns subjected to a 5mm load eccentricity can be seen to bend away from the heat source despite the load and moment acting in the same direction. This, as discussed in Section 4.6.4, is due to the low stress at the extreme fibre of the section which allows it to expand and increase bowing in the column however, as the thermal wave travels through the concrete to the slightly higher stressed material, transient thermal creep takes affect and the column begins to bend in the opposite direction. Due to the interesting differences between the 5mm and 25mm load eccentricities, an additional load eccentricity, 15mm, was investigated to try to determine the “critical load eccentricity” at which this transition occurs for this particular reinforced concrete column. It is important to note that all three of these tests were conducted under the exact same experimental conditions i.e. all three columns were loaded to 60% of their load capacity at ambient temperature, subjected to the same 70kW/m^2 heat flux using the same radiant panel, and heated on the “tension face”.

From Figure 4.18c, it is clear that the 15mm load eccentricity test behaves in such a manner that, when heated, it bows toward the heat source for the first 30 minutes of heating. It then reaches a plateau where, despite the temperatures continuing to rise within the column, no additional deflections are observed in the final 60 minutes of heating. This is a very clear indicator that there is a lot more to the response of concrete in fire than just the reduction in strength of the material itself. As discussed in Chapter 2, there are a number of mechanisms taking place within the concrete; reduction in strength, reduction in stiffness, thermal expansion of the aggregate, thermal contraction of the cement paste and transient thermal creep. All of this will contribute to the varied behaviour observed in Figure 4.18c. It is clear that, under this particular combination of load and moment, these factors balance each other outwith regards to the deflection histories of the columns. In order to design and accurately predict such behaviour in the event of a real fire, one must understand exactly how all of these mechanisms contribute to the overall behaviour of the column. Figure 4.18c is a particularly interesting plot to try to reproduce from a modelling perspective, something which is currently not possible without further investigation into the aforementioned material properties at elevated temperature.

For now one can speculate that, much like the 25mm load eccentricity, the load on the extreme fibre of the "tension face" of the column is insufficient for transient thermal creep to occur. This results in thermal expansion of the extreme fibre of the column for the first 20 minutes of heating, much like the 25mm load eccentricity scenario. However, after 50 minutes, the thermal wave within the concrete has travelled deeper into the section reaching the more highly stressed concrete. This concrete begins to compress as discussed in Section 4.6.4 and the column stops deflecting towards the heat source. It is at this point that the interesting behaviour begins. Between the time period of 50-90 minutes after the start of the test, the column is still being subjected to a $70kW/m^2$ heat flux and the thermal wave continues to travel through the section while the same load is applied. However, no additional lateral movement from the column is observed during this period. Therefore the combination of the aforementioned parameters are counteracting each other and stopping the column from experiencing greater deflections, despite the continuing damage the material is sustaining. If the column was to be heated for a period of more than 90 minutes, this behaviour would suggest that, much like the 5mm load eccentricity experiments, the bowing in the column would reverse and the column would eventually become "compression heated", should it survive for long enough.

4.6.6 Eccentricity and Heated Length

Changing the eccentricity of the load in combination with the heated length of the column will both change the load-moment combination that the column is subjected to and, the length of the column exposed to elevated temperature, therefore increasing the area of damaged concrete. Figure 4.19 illustrates the results of column tests conducted with differing load eccentricities and heated lengths.



(a) 5mm load eccentricity **(b)** 25mm load eccentricity

Figure 4.19: Midspan deflection of tests for C50 columns with differing heated lengths (a) 5mm and (b) 25mm load eccentricity

From Figure 4.19 it is clear that the columns subjected to a larger heated length behave in much the same manner as all of the columns discussed previously. In each case presented in Figure 4.19, the increase in the area exposed to elevated temperature results in a more severe response and magnifies the original response of the columns subjected to a smaller heated area.

"Compression heated" Specimens

Looking at the "compression heated" columns only, for both the 5mm load eccentricity and the 25mm load eccentricity, a similar but more pronounced deflection history was observed. For the 5mm load eccentricity case, when exposed to elevated temperatures on 33% of the columns length, the specimen failed roughly 80 minutes after the start of the heating phase while the heaters were still on. For the exact same loading scenario heated on 66% of the columns length, the exact same behaviour is observed but the column fails roughly 50 minutes after the start of heating. A full discussion of how the compression heated specimens behave in fire is provided on Section 4.6.3. As discussed in Section 4.6.3 the response of the columns is likely a result of transient thermal creep as a result of the elevated temperatures in the concrete and the high stress levels. Therefore, increasing the heated area, increases the length of the column over which transient thermal creep occurs, increasing the severity of the response and resulting in the column failing sooner. This is also true in the 25mm load eccentricity case where both compression heated columns survived the heating phase but, as a result of the detrimental effect of the cooling phase discussed in Section 4.6.3, failed after the heaters were switched off. As the column subjected to an increased heated length had more concrete capable of contracting in the cooling phase, the deflection of this column was more pronounced and failed more rapidly in the cooling phase than the columns exposed to elevated temperatures over a smaller area.

“Tension heated” Specimens

Much the same conclusion can be made for the “tension heated” columns in Figure 4.19a and Figure 4.19b however, some notable observations can be made in both the 5mm and the 25mm load eccentricity cases. As discussed in Section 4.6.5, the tension heated specimens subjected to elevated temperatures while loaded at 60% of their load capacity sometimes display a behaviour where they bend toward the radiant panel at the onset of heating before recovering and bending away from the heater as the heating phase progresses. This behaviour is discussed in Section 4.6.5. It can be seen in Figure 4.19a that the columns heated on the tension face display this same behaviour. The column subjected to heating over 33% of the columns length transitions at 50 minutes after the start of the heating phase. In comparison, the column heated over 66% of the length of the column transitions after only 10 minutes of heating. Beyond this point they behave like the "compression heated" specimens and columns subjected to a larger heated length failed almost immanently after the heat source was switched off.

In contrast to the 5mm eccentricity load case, the "tension heated" specimens exposed to elevated temperatures over differing heated lengths for the 25mm eccentricity load case behave in much the same manner. The main difference in the tests is that the column subjected to heat over a larger area experiences slightly larger deflections in the heating phase, and slightly larger contractions in the cooling phase. As discussed in Section 4.6.4, the stress at heated face for the "tension heated", 25mm load eccentricity case will be zero because the columns are loaded at the very edge of the middle third of the section. Therefore, it is unlikely that transient thermal creep will occur as it does with the 5mm "tension heated" load case or the "compression heated" columns. Therefore, the deflections in the columns will mainly be a result of the thermal expansion/contraction of the concrete, resulting in similar behaviour.

4.7 Magnitude of Load

4.7.1 Significance of Magnitude of Load

The magnitude of the load being applied to the columns may result in a number of interesting responses based on the material properties of concrete in the literature, all of which are discussed in Chapter 2. As discussed in Chapter 2, concrete responds differently during the heating phase of a fire depending on the stress distribution and temperature within. This phenomenon, called transient thermal creep, results in non-recoverable shortening of the concrete column and is of particular interest in this experimental test series because, in a reinforced concrete column subjected to both axial load, moment, and a non-uniform temperature gradient, the degree of transient thermal creep within the column will also be non-uniform through the section.

As already noted, one potential limitation of this test series is that the magnitude of the load imposed on the columns was applied as a percentage of the total strength of the column under a specific load condition rather than maintaining the same load between tests. The load applied to the columns was 60% of the maximum load achieved in ambient temperature conditions using the same structural arrangement. The experiments conducted at ambient temperature to determine the strength of the columns under each load arrangement have been detailed in Section 4.2. Therefore, each case of load eccentricity is subjected to a different axial load, limiting the ability to directly compare experiments at different eccentricities due to the fact that they have been stressed to different extents. However, as the load, eccentricity and deflections were monitored at all times during the experiments, it is possible to draw meaningful conclusions from the response of each column and use the data to validate modelling.

In addition to the tests subjected to 60% of their ultimate load at ambient temperature, each test was repeated applying minimal load (10kN) to the column under each arrangement. The purpose of this second set of tests was to determine:

1. the effect of the non-recoverable deformation of the column upon cooling back to ambient temperature as a result of transient thermal creep; and
2. if the load applied during the heating/cooling phase has any effect on the residual performance of the column, see Chapter 5.

4.7.2 Magnitude of Load and Concrete Strength

Looking at the performance of the columns in response to changing the magnitude of load applied in combination with the concrete strength used in the construction of the columns will have two main effects on the test. The load applied, one of the main contributing factors in transient thermal creep will be changed, and the material properties of the concrete may be slightly different as a result of the differing concrete mixes. Both of these parameters may have a significant effect on the response of the structure to elevated temperatures. For a full discussion on the tests conducted changing the magnitude of the load on the column in combination with the strength of the concrete used in the construction of the columns, see Section 4.5.2 and Section 4.6.3.

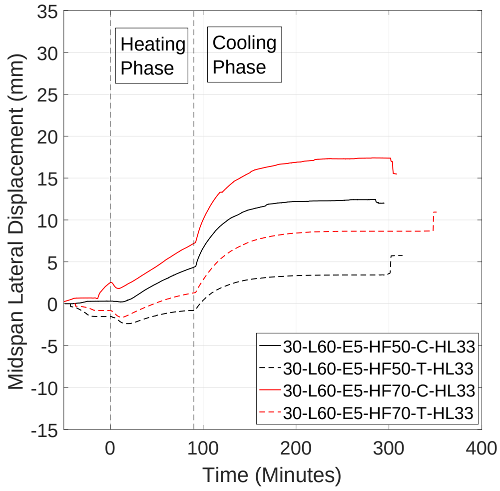
It was concluded from Section 4.5.2 and Section 4.6.3 that changing the magnitude of load has a large effect on the response of the columns during the heating and cooling. The difference in the response of the 30MPa columns and the 50MPa columns was however less significant. The columns of different strengths deflected to differing extents, but this would be expected not only from the different material properties of the concretes used, but also as a result of the different loads applied to the column, as discussed in Section 4.7. Despite the different levels of deflection experienced between the two types of concrete used, the general behaviour experienced is very similar i.e. "compression heated" specimens experience transient thermal creep resulting in deflection away from the heat source and the "tension heated" specimens bow towards the heat source due to thermal expansion of the extreme fibre on the heated face.

4.7.3 Magnitude of Load and Eccentricity

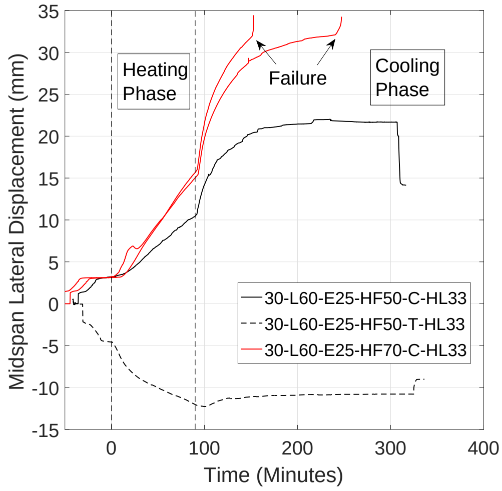
As discussed previously, the load applied to the columns for each eccentricity is a function of the maximum load in any specific arrangement. As a result, tests with a 60% load magnitude and different load eccentricities will have a different axial load applied, additionally changing the moment applied to the columns for each condition. It is therefore difficult to draw direct comparisons between the tests for these two parameters however, the trends and general behaviour may be determined and, most importantly, data is available for validation of the models described later in Chapter 6. For a full discussion on the response of the columns when changing the magnitude of load in combination with the load eccentricity, see Section 4.6.3.

4.7.4 Magnitude of Load and Heat Flux

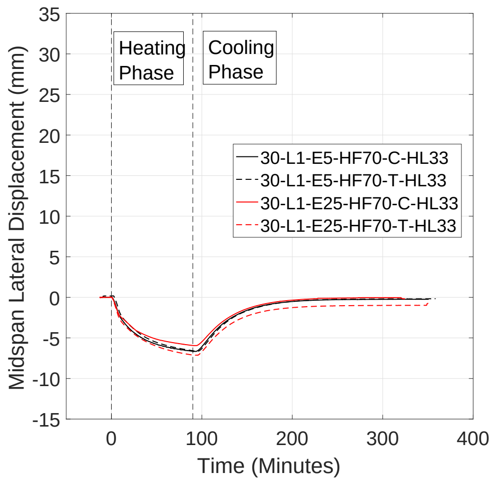
Through the experimental stage of this thesis, it was decided that the low load, low heat flux condition would not be conducted. This decision is based on the fact that the current literature suggests that, unloaded, no interesting behaviour would be observed and the columns simply behave in a similar fashion to the low load, high heat flux tests (with less severity). Taking this into consideration, Figure 4.20 illustrates the results of the tests conducted changing the magnitude of the load in conjunction with the heat flux the column is exposed to.



(a) 5mm load eccentricity, C30



(b) 25mm load eccentricity, C30



(c) Low load, 70kW/m^2, C30

Figure 4.20: Midspan deflection of tests at differing heat flux for (a) 25mm load eccentricity, C30 (b) 25mm load eccentricity, C30 (c) low load, 70kW/m^2, C30 concrete columns

Taking another look at the unloaded columns (Figure 4.20c), as discussed in Section 4.5.2, the columns exhibit the same behaviour in each case of load eccentricity and side heated (deflecting towards the heat source to a similar degree). The eccentricity of the load and side heated are both inconsequential with regard to the reaction of the columns to elevated temperatures when little or no load is applied. As no data has been gathered for the low load, low heat flux condition, it is not possible to make direct comparisons. However, as previously stated, it is expected that when exposed to a heat flux below 70kW/m^2, the columns will exhibit the same behaviour as in Figure 4.20c. The reduced temperatures in the column will however result in a less severe temperature gradient through the column, resulting in reduced thermal expansion, therefore reducing the deflections experienced by the column on exposure to elevated temperatures. It can therefore be concluded that, when loaded to a relatively low level compared with the capacity of the column (∼5%), the most significant driver with regard to the structural response of the column is the severity of the exposure to elevated temperatures and therefore the thermal gradient through the column. Higher temperatures/longer exposures will result in steeper/deeper thermal gradients respectively, both increasing the degree of thermal expansion within the concrete and worsening the deflections observed.

For the loaded columns however (Figure 4.20a and Figure4.20b), it is clear that a pattern is emerging with regard to the behaviour of the columns subjected to elevated temperatures under a sustained load of 60% of their ambient temperature capacity. The response of the columns subject to sustained load has been described in detail previously in Section 4.6.4. The mechanisms of response during heating and cooling are discussed and it is concluded that, as with the low load condition, an increased heat flux results in a less favourable response of the columns.

This conclusion does however come with a rather pertinent caveat. It has been observed in all of the tests conducted that an increased heat flux results in a greater deflection history of the reinforced concrete column. However, upon investigating the other relevant parameters (specifically load eccentricity), an interesting behaviour was observed (Figure 4.18c) in that a column subjected to 60% load at an eccentricity of 15mm appeared to stop deflecting while the internal temperatures continued to rise. The mechanics behind this response are discussed in Section 4.6.5. As is clear in Figure 4.20, the high load and low load conditions deflect in opposite directions and, as is clear from Figure 4.18c, there is a critical point at which this transition occurs. Looking at the "tension heated" specimens in Figure 4.20a, the heat flux required for this transition is clearly smaller than 50kW/m^2 (the smallest heat flux tested). Therefore, in the "tension heated" condition, it is conceivable that, although not observed in these experiments, a smaller heat flux could result in a less favourable deflection history during heating due to the fact that a reversal in the deflection of the columns may not occur, resulting in higher secondary moments. This does not however imply that the column will fail sooner. As a result of the lower temperatures in the concrete, the material properties will be more favourable and the element will be mechanically stronger. This would suggest that, despite the larger deflections during the heating phase, the columns subjected to a smaller heat flux are still residually stronger than the columns subject to a higher heat flux (see Chapter 5 for more details).

4.7.5 Magnitude of Load and Side Heated

As discussed in Section 4.6.3 and Section 4.7.4, increasing the load on the columns significantly changes their behaviour during the heating and cooling phases of the experiment. In addition to this, it is clear from Section 4.6.5 that heating the "compression face" or the "tension face" also has significant ramifications for the behaviour of the columns in terms of the direction of the displacement (towards or away from the heat source). As is clear from the remainder of Chapter 4, columns subjected to a low load experience deformations almost exclusively from thermal expansion of the materials. This changes as load is applied when, as discussed numerous times throughout this chapter, different levels of load induce differing degrees of transient thermal creep.

Compression Heated Columns

In the "compression heated" condition, the columns are subjected to elevated temperatures on the most highly stressed section of the column. As discussed in Section 4.6.5 and Section 4.6.4, this results in the columns experiencing transient thermal creep on the heated face of the column which results in the column deflecting away from the heat source. The extent of the transient thermal creep and the magnitude of the deflections experienced depends greatly on the magnitude of the load. As the magnitude of the load increases, the compressive stresses within the column increase, resulting in greater deflections due to transient thermal creep. See Section 4.6.3 and Section 4.7.4 for a full explanation.

Tension Heated Columns

In the "tension heated" condition, the columns are subjected to elevated temperatures on the lesser stressed section of the column however, as the maximum load eccentricity applied is 25mm (edge of the middle third) it should be noted that the "tension face" still experiences compressive stresses. The compressive stresses induced are however not enough to promote enough transient thermal creep in the concrete for it to overcome the effect of thermal expansion. In the instance that the columns are exposed to elevated temperatures, the column therefore bows toward the heater. The compressive stresses within the concrete are greater closer to the "compression face" therefore, as the thermal wave penetrates deeper into the concrete, the column begins to experience transient thermal creep to a greater extent as a result of the increased temperatures. When this occurs, the column appears to correct itself, transitioning from “tension heated” to “compression heated” and begins to deflect away from the heat source. This phenomenon has been discussed in greater detail in Section 4.6.5 and Section 4.6.4.

4.7.6 Magnitude of Load and Heated Length

As part of this experimental test program five experiments were conducted on columns where the heated length that the column was exposed to was doubled from 1/3 of the length of the column (467mm) to 2/3 of the column length (933mm). The purpose of this, as discussed further in Section 4.10.1, is to investigate the effect of an increased exposure area on the reinforced concrete columns under the same loading conditions as the remainder of the experiments. As there was a finite number of columns cast to complete this experimental test series, low load experiments with larger heated length were not conducted. As a result of the conclusions of the experiments conducted in Section 4.5.2, that the behaviour of the unloaded columns is primarily a result of the thermal expansion of the concrete, these tests were omitted from the experimental test series.

As low load - increased heated length experiments were not conducted, it is not possible to draw direct conclusions of what happens when increasing the heated length for unloaded columns. However, considering the behaviour of the remainder of the columns, it is likely that increasing the heated length of the unloaded columns would result in the columns deflecting further towards the heater during heating as a result of the increased area experiencing thermal expansion due to elevated temperatures. As the columns cool back to ambient temperature, the deflections would also reduce, trending towards zero however, it is likely that the columns subjected to higher moments would retain some residual deflections as a result of the increased compression stain within the concrete, potentially resulting in some slight non-recoverable shrinkage of the concrete due to transient thermal creep.

It is however possible to draw conclusions on increasing the heated length of the column when the column is subjected to a higher load level. For a full discussion on the result of increasing the heated length of the column in conjunction with the remaining parameters, see Section 4.10.

4.8 Heat Flux

4.8.1 Significance of Heat Flux

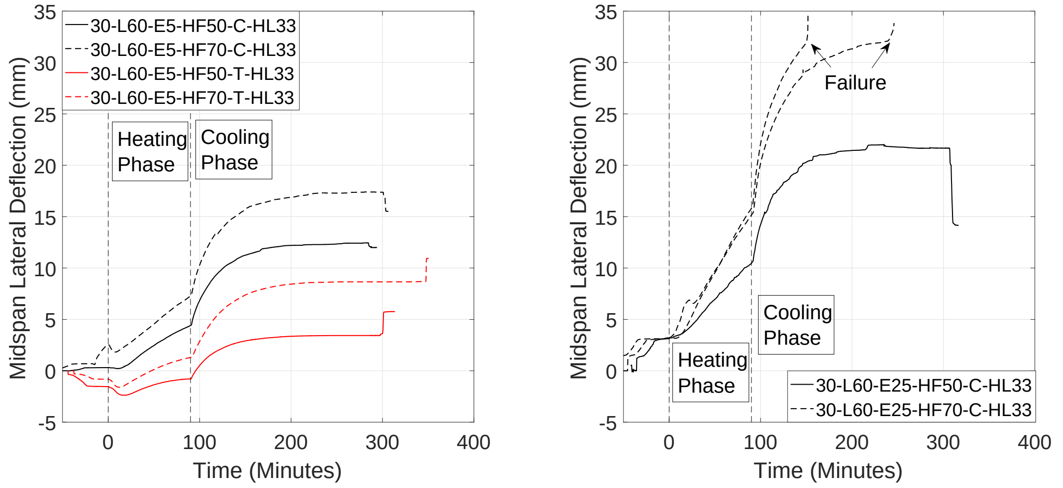
The heat flux refers to the total energy per unit of time applied to the surface of the column during the heating phase of the experiment. It holds significance in real fire scenarios due to the fact that it is difficult to precisely determine the heat flux on a concrete, or indeed any, element during a fire. Within this experimental test series however, the behaviour of "real fires" is outwith the scope of investigation and it is important to note that in any structural arrangement, the heat flux does not directly determine the response of the structure. The heat flux is the boundary condition which results in a temperature gradient within the concrete. It is ultimately the nature of this temperature gradient which determines the response of the structure. This is an important distinction, because there are many heating arrangements and temperature-time curves, which could ultimately result in the same temperature gradient through the column and therefore, the same structural response. As a result, the classic temperature-time curves used within "classical" standard fire resistance testing have not been considered herein. It was instead intended that a constant incident heat flux will be delivered on the surface of the columns to achieve different temperature gradients and levels of thermal “damage” throughout. Two values of the incident heat flux were used in conducting these experiments:

- $50kW/m^2$; and
- $70kW/m^2$.

It should however be noted that, upon conducting the first set of experiments on the weaker 30MPa concrete columns, the lower value of heat flux ($50kW/m^2$) did not induce significant damage on the concrete columns. As a result, the higher strength concrete columns were only subjected to the higher level of heat flux ($70kW/m^2$) in an attempt to draw more meaningful conclusions from the remaining experiments. For a full explanation of the reasoning behind this decision, see Section 4.8.2.

4.8.2 Heat Flux and Concrete Strength

Experiments were conducted on columns constructed with different strengths of concrete and, as discussed in Section 4.8.1, the columns were subjected to two differing heat fluxes. It is however important to note that only the weaker 30MPa concrete columns were exposed to the lower value for the heat flux ($50kW/m^2$). When initially conducting the experiments on the weaker concrete columns, columns were tested at both levels of incident heat flux. Figure 4.21 details the response of the weak concrete columns to both the $50kW/m^2$ and the $70kW/m^2$ heat flux.



(a) 5mm load eccentricity **(b)** 25mm load eccentricity

Figure 4.21: Midspan deflection of tests on the 30MPa concrete columns at differing heat fluxes for (a) 5mm and (b) 25mm load eccentricity

In every load case investigated, it can be concluded that increasing the heat flux the columns are subjected to produces a similar, but more severe, deflection history. All columns that deflected away from the heat source, deflected further away from the heat source when increasing the heat flux. It is clear from Figure 4.21b that there are no results for the "tension heated", 25mm load eccentricity tests. This is a result of a power failure close to the end of the test that resulted in a complete loss of data. Anecdotally, the author can however confirm that the columns behaved similar to that of the "tension heated" samples in Figure 4.18d, and the test conducted at a larger heat flux qualitatively experienced greater deflections during the heating phase. This result further confirms that increasing the heat flux exacerbates the behaviour of the columns and, in addition to the "compression heated" specimens, all "tension heated" columns that deflected towards the heat source, deflected closer to the heat source. It is also true that the columns that experienced the more interesting response (5mm load eccentricity, "tension heated") experienced the reversal in the direction of movement sooner than that of the lower heat flux.

It can therefore be concluded that, for the weaker set of reinforced concrete columns, increasing the heat flux that the columns are exposed to results in a similar, but more pronounced deflection history. This occurs because, with an increased heat flux, the transfer of energy into the concrete column happens more quickly. The temperatures within the columns will have a similar behaviour for the same loading/heating condition i.e. hot at the front and cold at the back, with the difference that the heated surface will attain higher temperatures. As a result the columns will behave in the same fashion due to the same stress state and similar but more pronounced temperature profile, but the column will deflect more rapidly/fail sooner on account of the weaker concrete.

Due to the conclusion of the experiments conducted on the weaker reinforced concrete columns, similar experiments were not conducted on the stronger reinforced concrete columns due to the fact that similar conclusions were likely. In place of differing the heat flux for the stronger reinforced concrete column, the heated length was increased from 467mm to 933mm. For full details of this and a discussion of the results and conclusions, see Section 4.10.

4.8.3 Heat Flux and Eccentricity

In changing the heat flux in combination with the load eccentricity, it would be expected that increasing the eccentricity would result in more pronounced deflections of the reinforced concrete columns. In reality the behaviour of the columns is more complex and the combination of heat flux and load eccentricity cannot be simplified in this manner. This is due to the fact that the side heated also plays an important role in this scenario. Increasing the heat flux or load eccentricity for a “compression heated” column has a different effect of doing the same for the “tension heated” columns. For a full discussion exactly how these parameters interact with each other, see Section 4.6.4 where effect of changing the combination of heat flux in conjunction with the load eccentricity has previously been discussed.

The simple conclusion that as heat flux is increased the deflections in the column also increase cannot be made. However, what can be concluded is that the general behaviour of the columns for the higher heat flux is the same as that of the lower heat flux, with an exaggerated response. The behaviour is simply experienced earlier than that of the lower heat flux experiments. For a full explanation, see Section 4.6.4.

4.8.4 Heat Flux and Magnitude of Load

As previously discussed in Section 4.7.4, altering the magnitude of the load that the columns are subjected to has a profound effect on the behaviour of the columns during the experiments. Under a minimal load, the response of the columns is primarily a result of the thermal expansion of the concrete and steel within. As a result, most of the deflections experienced are recovered on cooling. This is in comparison to the reinforced columns loaded to 60% of their ambient temperature capacity in which considerably different behaviour is observed. This is due to the properties of concrete when it experiences elevated temperatures while also being stressed to a relatively high level. This combination of high load and elevated temperatures results in transient thermal creep within the concrete which, depending on the specific load/heating condition, results in different responses during the heating/cooling phases of the experiments.

Much like Section 4.8.3, a simple conclusion on the response of the columns when altering theses parameters is not possible. Instead, it is clear that the behaviour of the columns at each level of load is exaggerated under higher levels of heat flux. This is a result of the magnified temperature gradient within the columns. For a full discussion of the effect of changing the heat flux in combination with the magnitude of the load applied to the columns, see Section 4.7.4.

4.8.5 Heat Flux and Side Heated

As discussed in Section 4.6.5, and in further detail in Section 4.9, changing the side heated has arguably the largest effect on the reinforced concrete columns of all of the parameters investigated as part of this experimental test series. As the side heated, and ultimately the magnitude of the stress on the heated face is so important in the response of the columns during and after exposure to elevated temperatures, a discussion of the effect of changing the side heated in conjunction with the heat flux has been provided when also discussing the effect of changing the eccentricity of the load, in Section 4.7.4.

It has been concluded from Section 4.7.4 that the side(s) heated will have a large effect on how the reinforced concrete columns may react as a result of being exposed to elevated temperatures. As discussed in detail in Section 4.6, the “compression heated” columns bow away from the heat source and the “tension heated” columns bow toward the heat source. Therefore, altering the heat flux for these scenarios will have a different effect on the response depending on the side heated. It has however been concluded that, in increasing the heat flux that the columns are subjected to, despite not directly increasing deflections, will magnify the response of the columns i.e. where the column deflects away from the heat source, it will deflect even further under a high heat flux or; where the bending moment transitions from positive to negative, increasing the heat flux will result in the transition occurring sooner in the experiment due too the increased speed of the thermal wave travelling through the section of the column.

4.8.6 Heat Flux and Heated Length

As part of this experimental test series, the effect of heat flux on the response was conducted on the weaker (30MPa) concrete columns. When conducting the experiments on the higher strength (50MPa) reinforced concrete columns, only the higher heat flux was used. This is due to the fact that the lower heat flux would likely not cause enough damage to the columns for any interesting conclusions to be made. In addition to this, it is likely that a similar conclusion would be likely i.e. the higher the heat flux, the more exaggerated the response of the columns. Therefore in the place of low heat flux tests, increased heated length tests were conducted on the remaining strong columns. As these parameters (heat flux versus heated length) were not investigated directly in comparison with each other, no conclusions can be made on the effect of changing these parameters on their effect on the response of the reinforced concrete columns.

As is concluded from the remainder of this section (4.8), increasing the heat flux that the columns are exposed to exaggerates the response of the columns. A similar conclusion can also be made when increasing the heated length of the column (see Section 4.10).

Increasing the heat flux that the columns are exposed to will result in higher temperatures being experienced, which is of no great significance due to the fact that concrete is rendered almost entirely structurally useless beyond 500^{o}C [4]. What is however more significant from a structural response perspective is that the thermal wave within the concrete will travel through the section more rapidly and create a larger thermal gradient. Therefore the behaviour experienced at smaller heat fluxes will be experienced sooner in the experiment and, as discussed earlier, the behaviour of the column will be exaggerated. On the other hand, when increasing the heated length of the columns, the thermal wave will travel at roughly the same speed through the section and the maximum temperatures achieved will be compatible. However, the increased heated length will result in a greater volume of concrete experiencing elevated temperatures which, much like increasing the heat flux, will exaggerate the previous behaviour experienced by the column in the smaller heated length scenario. It can therefore be speculated that the same conclusions would apply should these parameters be changed in combination with one another.

4.9 Side(s) Heated

4.9.1 Significance of Side(s) Heated

The side(s) heated refers to the location of the heat flux on the column. Within this series of tests, the columns were subjected to localised heat flux in the middle third of the column. As an axial load and a uni-axial moment was applied to the columns, the columns will undergo some curvature. In the condition of pure bending, classically, one side of the column would be in tension and the other in compression. As an axial load has been applied to the columns in these experiments to induce a bending moment (as with most column arrangements), this is not strictly the case. As discussed in Section 4.6, the side of the column referred to as the "tension face" may ultimately be experiencing compressive stresses due to the combination of axial load and being moment within the column.

The side heated is significant because, as previously discussed, concrete has different behaviour at elevated temperatures under differing levels of stress. In altering the side heated, the behaviour of the columns under these differing levels of stress was tested.

Three scenarios for the side(s) heated have been considered within this tests series:

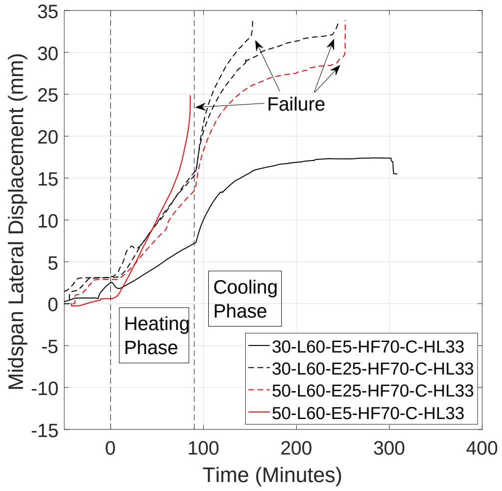
- compression face heated;
- tension face heated; and
- both the tension and compression face heated.

Due to the arrangement of the testing frame constructed, and the location of the radiant panels during the heating phase of the experiments, it was not possible to measure the deflections of the double sided heating scenario using the linear potentiometers used throughout the remainder of the tests described in this thesis. As a result, the deflections were obtained using digital image correlation software "GeoPIV-RG" [90] in conjunction with additional image analysis scripts developed "in-house" to measure displacements. For full details on the technique used and errors, see Section 3.2.5.

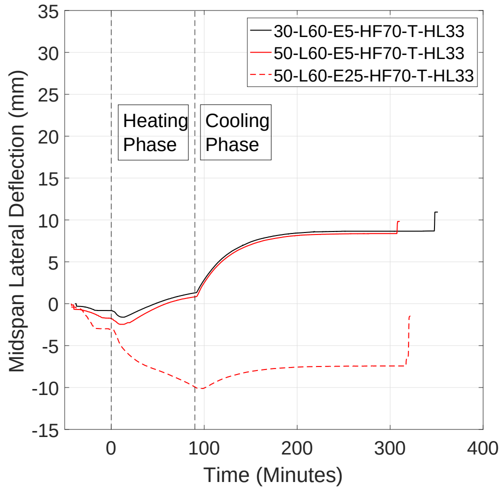
4.9.2 Side Heated and Concrete Strength

Concrete strength is not a simple parameter that can be changed with the ease of others investigated as part of this experimental test series. The only way to change the strength of the concrete, is to use higher strength concrete which will by its very nature, change the mix design of the concrete. As discussed in Chapter 2, changing the mix design of concrete can have significant effects on the behaviour of the material itself. Not only because the strength and stiffness of the material is different, but because the reaction of each specific concrete mix will have a unique response to elevated temperatures in terms of thermal expansion, the rate energy is absorbed and conducted through the material, the rate of decay of the material at certain temperatures in terms of strength and elasticity, and the magnitude of phenomena like transient thermal creep. Therefore, when comparing the results of the tests on the high strength concrete and the low strength concrete, it is important to keep these parameters in mind in the discussion because two concrete mixes of exactly the same nominal strength may react differently under the same test conditions as a result of having differing constituent mix proportions/materials.

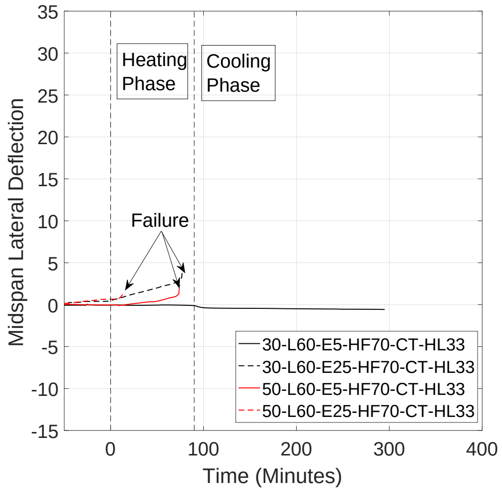
See Figure 4.22 for details of the comparison made when altering the side heated in conjunction with the concrete strength/mix.



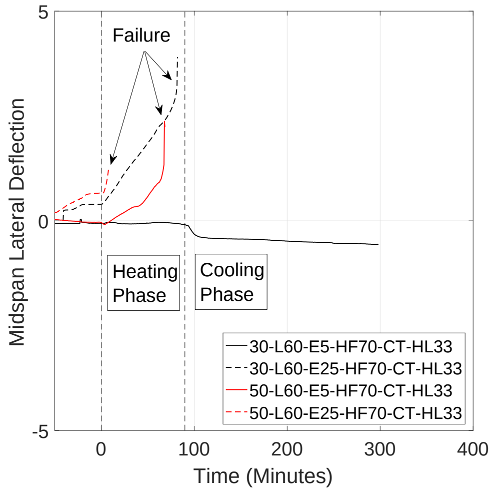
(a) "Compression heated" specimens



(b) "Tension heated" specimens



(c) "Double heated" specimens



(d) "Double heated" specimens - close up

Figure 4.22: Midspan deflection of tests at differing concrete strengths for (a) "Tension heated" (b) "Compression heated" (c) "Double heated", and (d) "Double heated" (close up) specimens

As discussed in previous sections, and as can be seen in Figure 4.22, changing the side heating has a significant affect on the response of the columns during the heating and cooling phases of the tests. For a full discussion on how changing the side heated tends to effect the response of the columns, and why this occurs, see

Section 4.6.4 and Section 4.6.5. It is clear that the trends discussed previously are true for both the 30MPa concrete and the 50MPa concrete in Figure 4.22. For all of the comparable tests conducted between the two tests, the reinforced concrete columns react in a similar manner i.e. the "compression heated" specimens undergo a large degree of transient thermal creep during the heating phase which results in the columns bowing away from the heat source, this is then exacerbated during the cooling phase from the contraction of the cooling concrete. Similarly the "tension heated" samples undergo thermal expansion with little transient thermal creep experienced due to the low stress levels of the concrete exposed to heat, and the columns deflect toward the heat source. Therefore, despite the differing values of the deflections experienced it is clear that, in terms of the general behaviour of the columns, changing the strength of the concrete has not had a huge effect on their response.

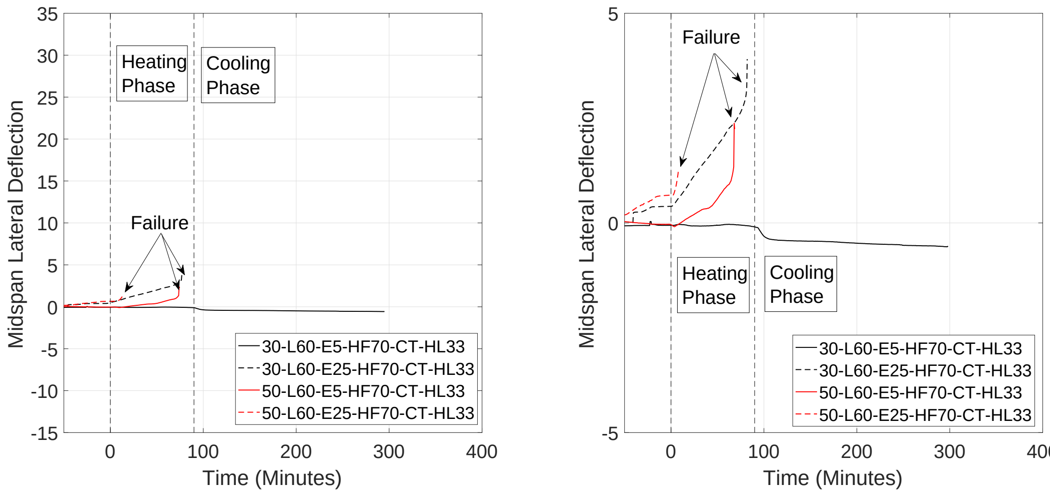
Looking at the columns heated on both the "tension face" and the "compression face" it is clear that it is slightly more difficult to draw meaningful conclusions because the change in concrete strength makes little difference in the behaviour of the columns. Every column exposed to heating on both sides experienced small lateral deflections during both the heating and cooling phases of the experiments. Considering the results of the tests conducted on the concrete columns subjected to single sided heating, this is to be expected due to the fact that the columns heated on opposing sides deflect in opposite directions. As discussed in Section 4.6.5, heating on either the "tension face" or the "compression face" individually resulted in deflections that increased the original mechanical deflections induced by the axial force. This is with the exception of the 5mm load eccentricity experiments that deflected in the opposite direction upon heating as a result of transient thermal creep. This does not occur in the 5mm load eccentricity experiments conducted on the columns heated on both sides. This is due to the fact that, similar to the other 5mm eccentricity tests, the stress of the "tension face" will be high enough for transient thermal creep to occur. However, as discussed in Section 4.6.1, the stress on the "compression face" will be greater. This has the effect of increasing the transient thermal creep experienced on the "compression face", thus making the columns behave like "compression heated" columns. As the deflections for all of the columns heated on both the "tension face" and the "compression face" are small, a close up of the graph has been provided in Figure 4.22(d).

4.9.3 Side(s) Heated and Eccentricity

As has been discussed previously in Section 4.6, changing the eccentricity of the load on the concrete columns with the same applied load will result in similar axial forces being applied, however the larger the eccentricity, the higher the moment applied to the column. As a result, changing the eccentricity of the load will alter the stress distribution within the section. To see how the stress distribution may change as a result of changing the eccentricity of the load applied to the columns, see Section 4.6.1. This change in the stress distribution through the section has a significant effect on the response of the reinforced concrete columns during heating and cooling. See Figure 4.18 for details of the experiments conducted changing the side heated with the eccentricity of the load applied.

As detailed in Section 4.6.5, there is a large difference in the behaviour of the columns for each of the combinations of load eccentricity and the side heated. These differences depend on the stress levels of the concrete exposed to elevated temperature. The higher the stress in the concrete, the more transient thermal creep experienced, resulting in greater deflections. Therefore, for the "compression heated" specimens, a higher load eccentricity results in higher stresses on the "compression face", making the column experience higher deflections. On the other hand for the "tension heated" samples, a higher load results in lower stresses on the "tension face", resulting in smaller deflections and an altogether more beneficial response. For full details of the experiments conducted altering the side heated in conjunction with the load eccentricity and a full discussion of the mechanics, see Section 4.6.5.

Figure 4.23 details the deflection response of the reinforced concrete columns subject to elevated temperatures on both the "compression face" and the "tension face" for differing load eccentricities.



(a) "Double heated" specimens

(b) "Double heated" specimens - close up

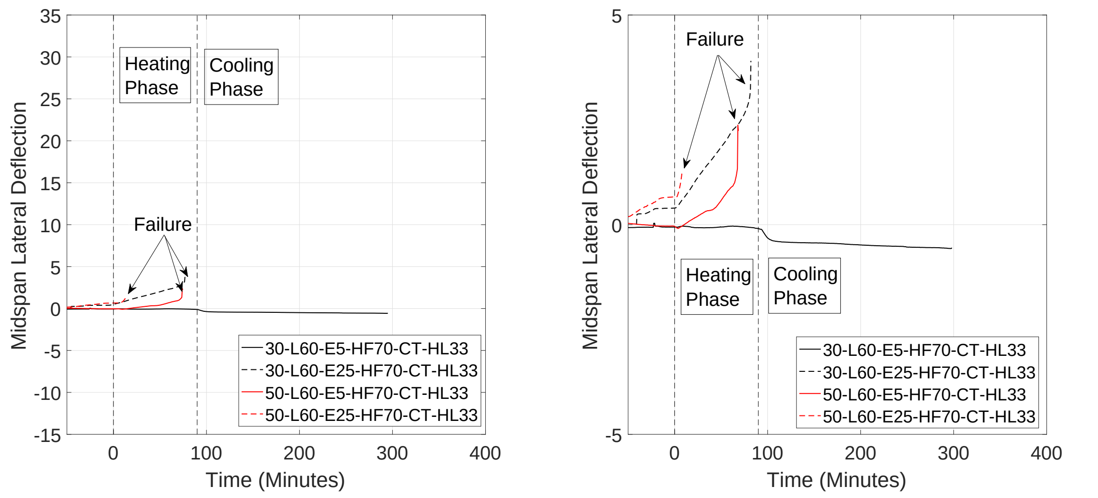
Figure 4.23: Midspan deflection of tests at differing load eccentricities

As can be seen from Figure 4.23, three out of four of the columns subject to double sided heating fail relatively soon after the onset of the heating phase. This is due to the fact that the damage sustained during this period is far greater than that of the columns heated on the "compression face" or the "tension face" only. When increasing the load eccentricity, similar to that of the columns heated on one side only, the deflection response of the columns is more significant and fails sooner than the 5mm eccentricity experiments. For a full discussion of why this occurs in relation to the secondary moments induced etc. see Section 4.6.5.

4.9.4 Side Heated and Magnitude of Load

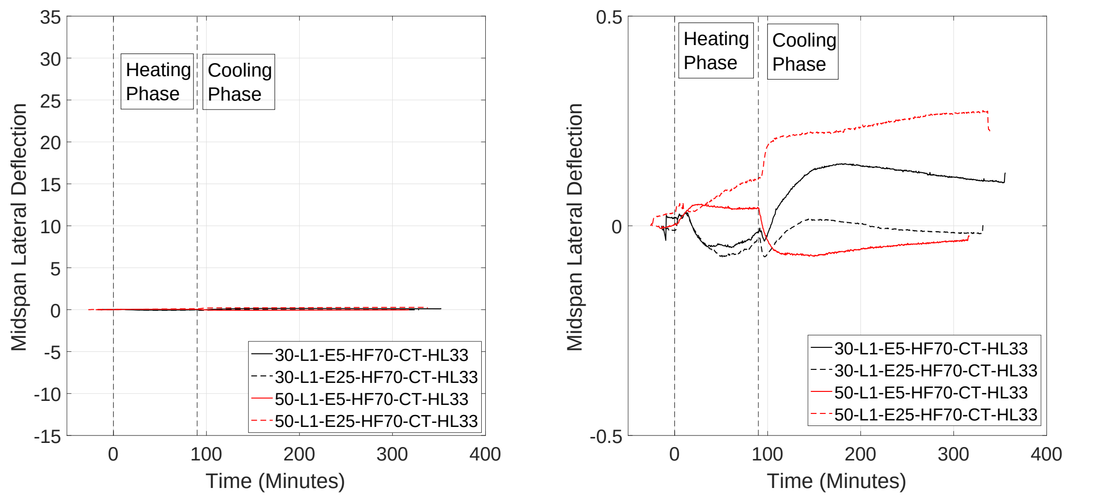
As discussed in Section 4.7, changing the magnitude of the load applied to the columns greatly effects their response during heating and cooling as a result of changing the stress distribution within the columns. It has previously been concluded that the columns will deflect almost entirely as a result of the thermal properties of the material under low load conditions. This changes under a higher load condition because the higher stress in the columns results in non-recoverable strains in the concrete known as transient thermal creep (see Chapter 2).

Figure 4.24 details the results of changing the side heated in conjunction with the magnitude of load.



(a) 60% of ambient load applied

(b) 60% of ambient load applied - close up



(c) 1% of ambient load applied

(d) 1% of ambient load applied - close up

Figure 4.24: Midspan deflection of columns heated on both faces for differing applied loads

As discussed in Section 4.6.3, under the low load condition all of the columns heated on one side only display a very similar behaviour. Each column deflects toward the heat source during the heating phase of the experiment, as described in Section 4.5.2. This deflection is then almost entirely recovered upon cooling back to ambient temperature. There are some slight differences in the magnitude of the deflections achieved in each of the experiments but all of the columns display very similar behaviour, no matter if it is “tension heated” or “compression heated”. The differences in the magnitude of the deflections is discussed in full detail in Section 4.5.2.

When the load applied to the columns is increased to 60% of its ambient temperature capacity, the conclusions discussed for the low load condition do not hold true. The increased stress in the section results in drastically different results depending on which side of the column is heated in relation to the eccentricity of the load. Under the “compression heated” scenario, as discussed in Section 4.6.4 and Section 4.6.5, the columns experience a relatively large level of transient thermal creep resulting in non-recoverable strains on the “compression face” of the column. This is then exacerbated during cooling when the deflections in the columns double from the values achieved during heating alone. The “tension heated” columns on the other hand behave in a similar manner to the columns subjected to a low load condition, with the exception that the deflections are not fully recovered during cooling. This is because, despite the fact that the concrete subjected to elevated temperatures is not stressed as highly as the concrete in the “compression heated” scenario, the columns will still experience a degree of transient thermal creep. For a full discussion on why this occurs and the difference between the columns subjected to elevated temperatures on the “compression face” compared to columns heated on the “tension face”, see Section 4.6.4 and Section 4.6.5.

As seen in Figure 4.24, when the columns are subjected to double sided heating on both the “compression face” and the “tension face”, the resulting lateral midspan deflections of the columns are small. As the columns have been heated on both sides, the concrete on both faces undergo thermal expansion. With the absence of any considerable load, the tendency of the “compression face” concrete to undergo transient thermal creep is minimised. Therefore the behaviour of the concrete on both sides of the column is almost identical, leading to very small displacements. Upon cooling, similar to the unloaded columns heated on one face, the deflections are almost entirely recovered when the columns cool back to ambient temperature conditions.

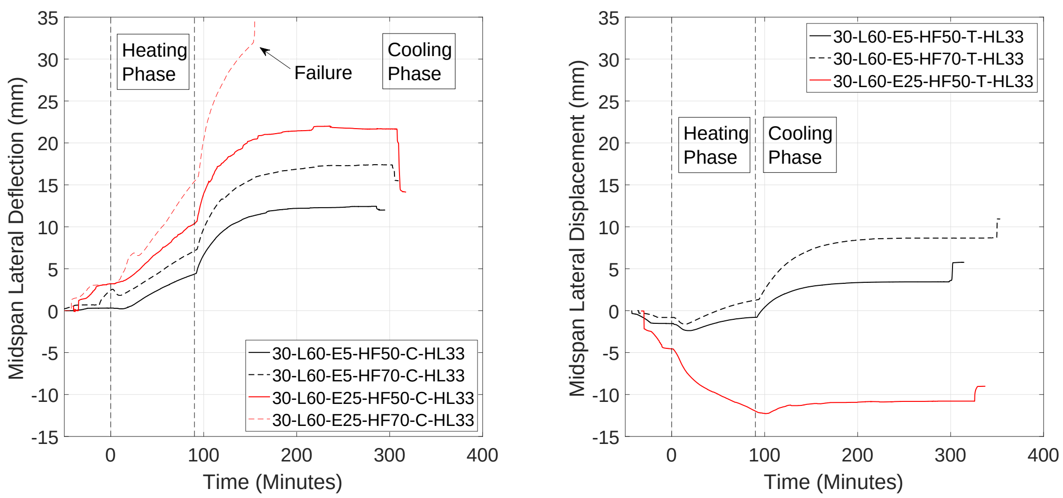
As the concrete columns heated on both the “tension face” and the “compression face” sustained a very large degree of damage, many of the experiments conducted on the columns under this condition failed during the heating process however, definite trends can be seen in their behaviour. As discussed in Section 4.9.2, each of the columns subjected to double sided heating deflect in such a manner that it increases the deflections of the column in the same direction as the mechanical load. This is a result of how the stress is distributed within the section when the columns are subjected to both moment and axial force and the differential transient thermal creep that will be experienced on both either side of the column.

It can therefore be concluded that, when changing the side heated in conjunction with the magnitude of the load, there are many factors that should be considered. The response of the columns will be dependent on the degree of transient thermal creep experienced during the heating phase of the experiment which is very sensitive to both the stress in the column and the temperature gradient through the depth of the columns. The degree of transient thermal creep will not only effect how far the column deflects, resulting in greater secondary moments, but also the direction in which the column deflects (towards or away from the heat source). If the concrete subjected to elevated temperatures is not stressed highly, thermal expansion will dominate, resulting in the column deflecting towards the heat source during heating, this will then be partially recovered during cooling. If the concrete subjected to elevated temperatures is stressed to a high level, transient thermal creep will dominate and the column will deflect away from the heat source during heating, and even further from the heat source during cooling.

This will result in greatly increased secondary moments and an increased chance of failure.

4.9.5 Side Heated and Heat Flux

The effect of altering the side heated with the heat flux applied to the columns was conducted on only the weaker 30MPa concrete columns. This was due to the fact that the stronger 50MPa reinforced concrete columns were used to investigate the effect of changing the heated length of the columns as opposed to the heat flux. Figure 4.25 details the results of changing the side heated in conjunction with the heat flux the columns are subjected to for the lower strength (30MPa) reinforced concrete columns.



(a) Compression heated specimens **(b)** Tension heated specimens

Figure 4.25: Midspan deflection of experiments on 30MPa concrete columns subjected to different heat fluxes for (a) "Compression heated" columns and (b) "Tension heated" columns

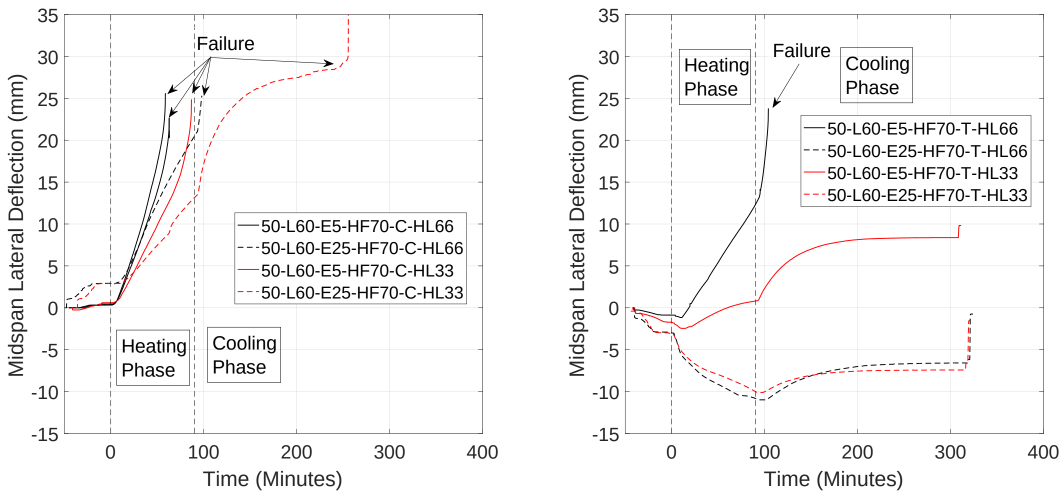
As is clear from Figure 4.25 all of the columns subjected to elevated temperatures on the same face in relation to the eccentricity of the load ("tension heated" or "compression heated") display the same behaviour with larger deflections for the larger heat flux. For a full discussion on the why the "tension heated" specimens deflect towards the heat source and the "compression heated" specimens deflect away from the heat source, see Section 4.6.4 and Section 4.6.5.

In increasing the heat flux it is clear that the behaviour of the columns is exaggerated, i.e. if the column deflected away from the heat source during heating, it deflects even further from the heat source with a higher heat flux. It is, however, clear that a test for a "tension heated" reinforced concrete column subjected to the higher heat flux of $70kW/m^2$ has not been detailed in Figure 4.25. This is due to a power failure at the end of the test which resulted in a total loss of all data for the test. Despite no actual data having been retrieved from the test, the author can, anecdotally, confirm that the behaviour of the column was similar to that of the experiment conducted with a lower heat flux, with the exception that the deflections achieved were greater. Confirming what was concluded earlier in this section, that the increased heat flux exacerbates the behaviour of the columns.

4.9.6 Side Heated and Heated Length

When investigating the side heating in combination with the heated length, it should be noted that only "tension heated" and "compression heated" experiments were conducted. Two sided heating with an increased heated length was not conducted as part of this experimental tests series as a result of the double sided heating tests conducted under the smaller heated length condition. Section 4.9.2 details the results of the double sided heating tests conducted. It was concluded from Section 4.9.2 that, given the heat flux being considered, the double sided heating scenario for the 1/3 heated length was enough to induce catastrophic failure of the columns relatively quickly in the heating phase. In addition to this, as detailed in Section 4.10, increasing the heated length of the column to 2/3 of the length of the column also greatly increases the damage sustained by the columns. Due to this fact, the combination of a 2/3 heated length and double sided heating was not conducted because damage sustained by the columns would likely be too severe, limiting the value of the tests.

"Tension heated" and "compression heated" tests were however conducted with an increased heated length. See Figure 4.26 for details of the tests conducted comparing these parameters.



(a) Compression heated samples **(b)** Tension heated specimens

Figure 4.26: Load-displacement response of columns subject to different sides heated and heated lengths

As is clear from Figure 4.26, for both the "tension heated" and the "compression heated" scenario, increasing the heated length of the columns magnifies the response of the columns. Similar to increasing the heat flux on the columns (described in Section 4.8), increasing the heated length of the columns results in a greater volume of concrete increasing in temperature. Therefore, as the concrete along the length of the column is stressed to relatively the same degree, the additional concrete exposed to elevated temperatures will behave in a similar manner as the concrete in the lower heated length scenario. As the behaviour of the concrete depends heavily on the stress distribution through the section, it is not possible to make a simple conclusion on what happens however, it is clear that the response of the columns is magnified as a result of increasing the heated length of the columns, i.e. where the column deflects away from the heat source, it will deflect further from the heat source with a greater heated length.

For further details on how changing the heated length of the columns alters their response, see Section 4.10.

4.10 Heated Length

4.10.1 Significance of Heated Length

The final parameter investigated experimentally as part of this thesis is the heated length of the column. Most of the columns in this test series were heated in the middle third of the column. The final tests conducted increased this to the middle two thirds of the column. It can be reasonably assumed that, as the heated length of the column is increased, the column will behave in a less favourable fashion. This is indeed the case, as will be discussed later in this section, however testing the columns in this fashion gives modellers the opportunity to further interrogate/validate their models under a greater number of conditions.

4.10.2 Heated Length and Concrete Strength

The effect of changing the combination of heated length with the strength of the concrete used in the construction of the reinforced concrete columns is one which has not been investigated as part of this experimental test series. Due to the fact that the heated length was only altered for the stronger (50MPa) reinforced concrete columns, it is not possible to draw any conclusions on how increasing the heated length of the columns affects its response in conjunction with the strength of the concrete used in the construction of the elements.

4.10.3 Heated Length and Eccentricity

A series of experiments was conducted investigating how changing the heated length of the column affects its structural response in conjunction with the eccentricity of the load applied to the column. It was concluded in Section 4.6 that the load eccentricity has the effect of altering the stress state within the section of the column due to the increased (or decreased) moment applied to the column by increasing (or decreasing) the eccentricity of the load applied. When also subjected to elevated temperatures this results in differing degrees of transient thermal creep which in turn results in interesting structural responses based upon the specific loading/heating condition the column is subjected to.

Altering the heated length of the column, on the other hand, results in an increased (or decreased) volume of concrete being exposed to elevated temperatures. In the case of the specific experimental test setup used as part of this experimental test series, as the stress distribution along the length of the column is similar at any point along the columns length, increasing the heated length of the column will magnify the response of the column with a smaller heated length as the additional concrete involved will behave in the same manner due to the similar stress state and temperatures it is exposed to. This is has been detailed further in Figure 4.27.

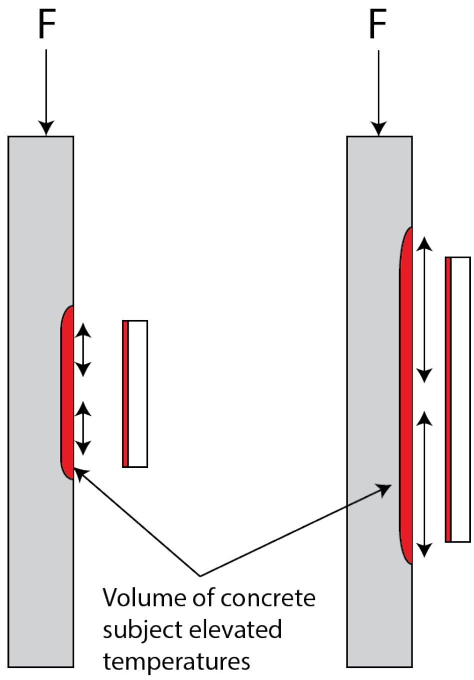


Figure 4.27: Illustration of the volume of concrete exposed to elevated temperatures for differing heated lengths

For a full explanation of the response of the columns subjected to differing heated lengths and load eccentricities, see Section 4.6.6.

4.10.4 Heated Length and Magnitude of Load

As part of this experimental test series, the effect of changing the heated length in combination with the magnitude of the load was not investigated. Only two load levels were considered during this test series (10kN and 60%). Due to the fact that the load level in the low load condition was so small, the response of the columns was dictated purely as a result of the thermal gradient of the columns (see Section 4.5.2). As a result, based upon the current knowledge of the behaviour of concrete at elevated temperature, it is likely that increasing the heated length of the column would succeed in exaggerating the results of the experiments conducted with a smaller heated length. Therefore the only experiments conducted with a larger heated length were those subjected to the higher load level. It is therefore not possible to draw direct conclusions on the effect of changing the heated length in conjunction with the magnitude of the load that the columns are exposed to.

4.10.5 Heated Length and Heat Flux

As part of this thesis, experiments investigating the effect of changing the heated length of the columns in conjunction with the heat flux were not conducted. As discussed in Section 4.8.6, an investigation of the effect of changing the heat flux on the structural response of the columns was conducted on the weaker (30MPa) columns. As the results of these tests indicated that the smaller heat flux experiments would not induce enough damage on the column to achieve a sufficient response upon which to draw conclusions. As a result, the impact of the heated length on the response of the columns was investigated for the stronger (50MPa) concrete columns.

As discussed in Section 4.8.6, both the heat flux and the heated length have a large effect on the volume of concrete being damaged, therefore in general for both parameters, increasing the value of heat flux or the heated length magnifies the response of the columns due to the increased volume of concrete being exposed to elevated temperatures.

4.10.6 Heated Length and Side Heated

When investigating the heated length of the columns, only the "tension heated" and the "compression heated" conditions were investigated. No double sided heating experiments were conducted for an increased heated length. Based upon the results of the double sided heating experiments and the increased heated length experiments, it is unlikely that the columns, given their current design and the loads applied, would produce sufficiently interesting results for these tests to be conducted due to the extreme damage induced by this particular heating scenario.

For a full discussion on the results of altering the heated length of the column in conjunction with the side heated, see Section 4.9.6. It was concluded from this investigation that increasing the heated length of the column resulted in a magnified response of the reinforced columns. As discussed previously, this does not necessarily mean that the column experiences greater deflections, rather that the general trend of the response is magnified, i.e. if the column deflects away from the column, increasing the heated length makes the column deflect further away from the heat source, and if the column begins to deflect in one direction but transitions and deflects in the other direction during the tests, this occurs sooner. For a full discussion of the results and the physical reasons for their responses, see Section 4.9.6.

4.11 Theoretical Real-World Example

Throughout this chapter, the heating/cooling results of the experimental test series conducted have been presented. This has primarily been presented in the form of a discussion of the deflection of the columns, temperature gradients within the concrete section, and loads/moments applied to the specimens. The purpose of these experiments is to determine how accurately the current methods used in design and analysis can be applied to non-standard conditions, such as the experiments outlined within this chapter. This section aims to give some context to the experiments conducted in terms of design of real buildings.

It is important to note that, as a result of the experimental setup, the displacement of the columns have been presented in two ways; towards the radiant panels (negative (−) in Figure 4.28), or away from the radiant panels (positive (+) in Figure 4.28). When conducting different experiments the test apparatus remains stationary and the reinforced concrete columns were moved into position to alter the heating/loading scenario. Therefore a "compression heated" column tested under the same loading condition as a "tension heated" column was rotated 180^o to heat the opposing face (See Figure 4.28).

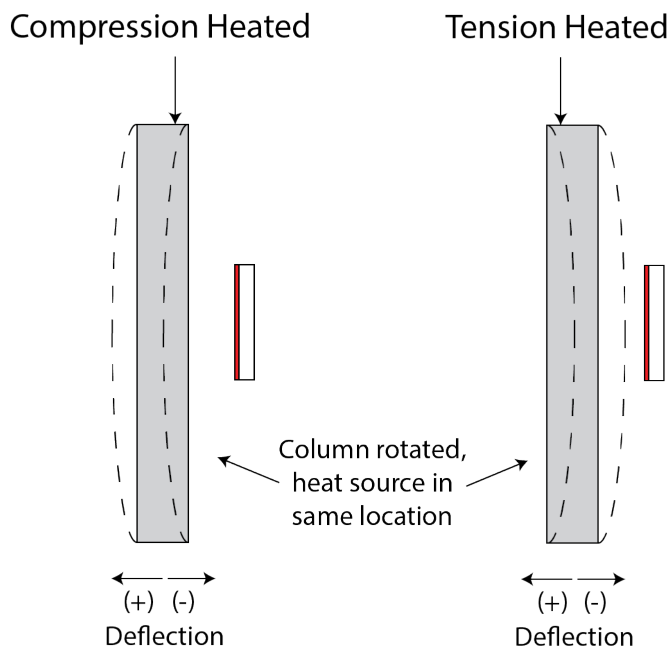


Figure 4.28: Test setup illustrating the relative displacement of "compression heated" and "tension heated" concrete columns

This has resulted in the behaviour of the columns being measured in relation to the heat source however, in a real fire situation, the deflections are not measured in relation to the location of the fire but in relation to the column itself. As a result, if the columns are considered to be identical with regard to load and orientation, and the heat source moved, the deflection of the columns would, for almost every test, be in the same direction. Figure 4.29 below illustrates this point.

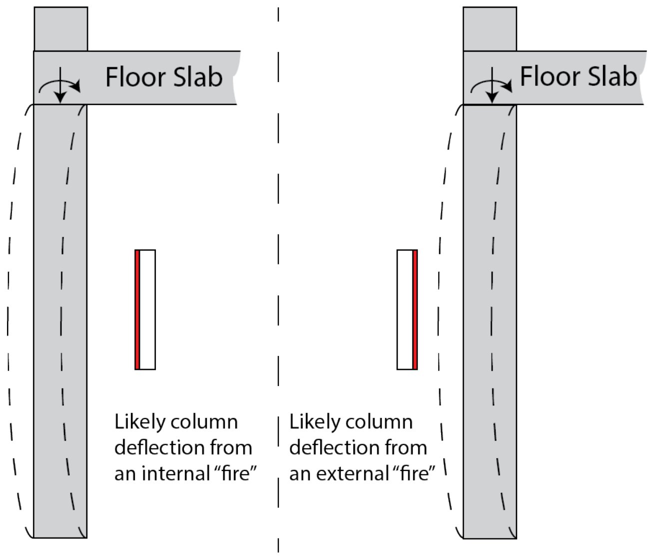


Figure 4.29: Illustration of the likely deflection of a reinforced concrete column due to an internal or an external "fire" based upon the experimental results of this test series

This gives more clarity when thinking about the response of real reinforced concrete structures and very clearly illustrates that, a simple calculation of the reduction in strength of the concrete and steel reinforcement is insufficient to capture the full element behaviour let alone full structural response. This becomes even more critical for columns that are likely to be heated on the compression face (edge columns) as this configuration results in large deflections during both the heating and cooling phase of the experiments conducted, and was the only configuration to consistently fail during the experiments. In addition, this brings into question the design methods detailed in BS EN 1992-1-2 [2]. These methods have been discussed in more detail and evaluated against the columns tested as part of this research project in Chapter 6.

4.12 Discussion of Results

Throughout this section there has been a large volume of data presented with regard to the tests conducted on the 46 reinforced concrete columns cast. The results have been broken down into sections discussing the effect of each parameter investigated and how it alters the structural response of the columns. It should however be noted that these parameters are simply boundary conditions that determine the internal stress distribution/thermal response within the column. It is ultimately this stress distribution that determines the behaviour of the columns. It is therefore not always entirely appropriate to assume that applying a load at a higher eccentricity will result in a column failing at lower load because there may be other external influences on the element such as differing thermal conditions and more beneficial or detrimental secondary moments as a result of the increased load. Instead, to determine the structural response at elevated temperature, it is necessary to first determine the stress distribution and temperature gradient within the section. As a result of the interesting material properties of concrete under heating and axial load, these properties ultimately determine the response. In order to accurately determine these properties however, it is necessary to fully understand the structural and thermal boundary conditions. The structural boundary conditions within a building may be determined with a relatively high degree of confidence however, the thermal boundary conditions prove to be a more challenging task for researchers and practitioners alike due to the unpredictability of fires within buildings. The prediction of accurate thermal boundary conditions resulting from a fire in a building is outwith the scope of this research project, therefore all discussions and analysis will neglect this fact and concentrate purely on predicting structural behaviour from the measured and quantifiable boundary conditions imposed during these experiments.

There are a number of possible approaches available to analytically/computationally determine the response of reinforced concrete columns under these conditions and determine the parameters described above. An analytical sectional analysis model has been developed to predict and quantify the response of the columns tested. Varying degrees of success have been achieved with this model. The merits and limitations of this approach have been discussed in detail in Chapter 6.

4.12.1 Significance of Parameters

A large volume of data and material regarding the results of the experiments conducted has been presented in this chapter. Due to the fact that such a large number of parameters were tested, great variation in the behaviour of the columns during the experiments has been observed. A brief summary is therefore provided in relation to the findings of the individual parameters tested:

Concrete Strength In changing strength of the concrete used in the construction of the reinforced concrete columns it is necessary to also change the concrete mix. Therefore not only the concrete strength was changed in investigating this parameter. As a result, as part of this research project, instead of drawing direct comparison between columns constructed with different types of concrete, it has been accepted that each concrete mix will have its own unique material properties and instead general trends between the two mixes have been commented on. Generally it has been concluded that the two concrete mixes used, despite altering the strength/stiffness of the concrete columns, they have displayed very similar behaviour when compared with the other parameters investigated. The "compression heated" specimens and "tension heated" specimens, despite achieving different absolute deflections, the response observed was almost identical. Therefore, the general behaviours of the two concrete mixes employed as part of this test series were very similar.

Load Eccentricity: As the eccentricity of the load applied to the columns increases, as does the moment induced by the eccentric load. Therefore, under ambient temperature conditions, a larger load eccentricity will result in larger deflections and a reduced ultimate capacity of the column. When exposed to elevated temperatures, a change in the eccentricity of the load will alter the stress distribution within the column. This change in stress distribution will ultimately determine how the column reacts. If the stress on the heated face is large enough to result in transient thermal creep, the column will deflect away from the heat source during both heating and cooling, otherwise the column will most likely deflect toward the heat source as a result of thermal expansion. Should the stress level be high enough to only slightly induce transient thermal creep, as in the 15mm load eccentricity case, the behaviour of the column will be highly dependent on other parameters such as the heated length and the heat flux, and may transition between positive and negative moments depending on the exact load configuration.

Magnitude of Load: Similar to altering the load eccentricity, changing the magnitude of the load applied to the columns will have an effect on the stress state within the cross section of the reinforced concrete columns. The axial load, in combination with the eccentricity of the load, will determine the stress distribution within the column which, in turn, will determine to what extent transient thermal creep will occur (if at all) through the depth of the section. For a full discussion of the mechanisms at play when changing the magnitude of load, see Section 4.7

Heat Flux: The incident heat flux on a structural element during a fire is one of the most import parameters designers are interested in, it is also arguably the most difficult to predict. Therefore, as discussed in Section 4.8, predicting the heat flux on an element in a real fire is outwith the scope of this thesis. Instead, values of heat flux have been selected and applied consistently for all of the experiments conducted. This is significant because the heat flux will determine the temperature gradient within the structural element and will have a significant impact on the structural response. Specifically within the boundaries of this thesis it was concluded that, not only did a higher heat flux result in a greater weakening of the concrete, but also in greater non-recoverable deformations as a result of increased differential transient thermal creep resulting from both the elevated temperatures and stress distribution through the section. See Section 4.8 for full details.

Side(s) Heated: In changing the side heated, much like altering the heat flux that the columns are exposed to, the temperature gradient through the column will change and the structural response may be vastly different, despite being exposed to the same heat flux for the same duration of time. This is due to the fact that in changing the side heated, the concrete exposed to elevated temperatures will be subjected to a different level of stress, resulting in what could be a very different structural response. It has been shown through this experimental test series that this alone can make the difference of the column surviving for the duration of the experiment, or failing catastrophically during the heating or cooling phase. Ultimately, this is considered in the current design guidance however, only for the application of standardised temperature-time curves and the material properties that are the most important (thermal expansion and the possibility of transient thermal creep) do not need to be considered in this calculation at all. This is also significant due to the fact that much of the current fire resisting testing is completed on elements heated on all four sides, and is terminated after the heating phase has completed. It has been shown during the experiments conducted that the cooling phase also holds great importance in determining the structural resistance of a column not least because the side heated will ultimately determine whether the element recovers during cooling or is damaged further. See Section 4.9 for full details and discussion of results.

Heated Length: The heated length is the final parameter that was changed to alter the temperature gradient within the column. Under the specific arrangement used in this experimental test series, increasing the heated length on one face of the column would increase the volume of concrete immediately exposed to the heat source. This concrete was stressed to a similar level as the concrete on the remainder of the face and therefore, increasing the heated length of the columns resulted in a magnified response of the columns compared to the experiments conducted with a smaller heated length.

It has been demonstrated throughout this chapter that predicting the structural response of a reinforced concrete column exposed to elevated temperatures while also loaded can be an incredibly challenging and complicated task for engineers. Through this section, the results of many tests have been presented and discussed with respect to every parameter that has been changed. It is however clear that, when altering one parameter, the response of the reinforced concrete columns can change drastically because ultimately, the response of the columns is determined by two parameters; the stress distribution within the section and; the temperature gradient within the column. Every parameter that has been investigated as part of this experimental test series has ultimately affected one of these two parameters within the concrete section itself.

When considering this for design, as discussed in Chapter 2, engineers have a relatively high degree of confidence when it comes to determining the stress state within the cross section. What is however more complicated is the determination of the temperature gradient in the section and the material response of the concrete to this specific combination of load and temperature. Investigating methods to determine the temperature gradient and material response of concrete is outwith the scope of this thesis, however the experimental test series conducted has indicated possible gaps in knowledge that should be investigated further to progress the state of the art to a point at which engineers can be more confident in this area. See Chapter 7 for full details of the possible next steps that, in the authors opinion, should be taken to progress the state of the art when analysing the response of reinforced concrete columns subjected to elevated temperatures for the heating and cooling phase of a fire.

Chapter 5

Experimental Results: Residual Response

5.1 Introduction

In Chapter 4, the results of the reinforced concrete column experiments were discussed in relation to their response during the heating and cooling phases. As detailed in Chapter 3, all of the columns that survived the heating and cooling phases of the experiments were allowed to cool back to ambient temperature conditions before being subjected to a destructive experiment at the same load eccentricity that was applied during the initial phase of the experiment. This Chapter discusses the results of the column experiments in relation to their ambient temperature response after cooling back to ambient temperature.

In addition to the residual strength experiments conducted, a non-destructive experimental method (ultrasonic pulse velocity) was used before destruction to attempt to determine the condition of the concrete and potentially predict how it may respond after cooling. As has been discussed previously in Chapter 2, after exposure to elevated temperature, concrete displays permanent changes to its material properties, and its response when loaded after a fire. Therefore, the ability to detect these changes non-destructively may provide some insight as to how a concrete structure subjected to elevated temperatures will react when loaded after returning to ambient temperatures i.e. a concrete structure is recommissioned after a fire. This could be very useful in post-fire condition assessment of concrete buildings in practice.

In real terms after a fire in a building, ideally the structure will still be capable of resisting the design loads that the structure is subjected to during daily operation. Therefore, a direct comparison is to be made to the ambient temperature response of the columns under the different load conditions in relation to their ambient temperature response after exposure to elevated temperatures. This will provide insights into how structures may respond to different heating conditions, but also how the same structural element will respond to the same heating conditions under a different structural arrangement.

5.2 Non-Destructive Testing

Some of this thesis has concentrated on using the data collected to develop tools and models (detailed in Chapter 6) to be used in the design of concrete structures and, specifically, reinforced concrete columns. The goal of these models is ultimately to determine the response of any reinforced concrete column during the heating phase of the experiments, and beyond, when the concrete begins to cool and after the column has cooled back to ambient temperature. This is an important area of investigation in the field of fire safety engineering, as it is essential to understand, at the start of the design process, how a building being designed may respond under various potential fire conditions. This is an area in which the volume of experimental data and theoretical understanding is currently limited. This section aims to outline the work completed as part of this experimental test series to investigate the application of these techniques in the assessment of fire damaged structures, discussed further in Chapter 2. The ultimate goal of undertaking any non-destructive testing with respect to fire damage is to accurately determine the temperature profile through the concrete section to estimate the residual mechanical properties of the concrete. This chapter discusses how this has been achieved with respect to the concrete columns investigated as part of this experimental test series.

5.2.1 Why Conduct a Non-Destructive Test

The primary reason for conducting any post-fire analysis on a fire damaged building is to ensure the safety of the occupants within the building should it continue to be used in some way. Is the structure safe and, if not, what reparations are required to achieve an adequate level of safety? This should always be the first consideration in any damage assessment conducted. However there are other benefits from an increased wealth of knowledge in this area. In today's socio-political climate, there is a huge investment in sustainability and sustainable construction. As this gathers speed and governments take note of climate change, this investment is only likely to grow in the coming years. This is especially topical in construction because, as an industry, it accounts for a total of 4-5% of the total worldwide CO_2 emissions [7]. Much of this is due to the production of cement powder. As a result, each building of primarily concrete construction contains considerable embodied energy. It may therefore be in the best interest, from a sustainability perspective, to conduct an in depth post-fire analysis of a structure to detail the reparations required, rather than demolish the building and replace it after a fire. This will also provide the added benefit that the downtime experienced by any business reliant on the building may be minimised as well as (possibly) the total cost of recommissioning.

5.2.2 Ultrasonic Pulse Velocity

In conducting the non-destructive testing experiments on the concrete columns, Ultrasonic Pulse Velocity (UPV) was measured using a Pundit apparatus [91]. As described in Chapter 2, the Ultrasonic Pulse Velocity method is one in which ultrasonic sound is sent from an emitter placed on one side of the concrete to a receiver placed on the other side of the concrete. The output from this method is simply the time it takes for the sound to travel from the emitter to the receiver, and is influenced by the elastic stiffness and the mechanical strength of the material being tested [69]. From these measurements it is possible to calculate the pulse velocity quite simply by using Equation 5.1.

$$\nu = \frac{D}{\Delta t} \tag{5.1}$$

Where:
v = velocity (m/s)
D = distance (m)
Δt = time (s)

Using this pulse velocity, it is possible to use empirical relationships between this, and certain material properties of the concrete. This method can be effective in determining whether concrete has been subjected to elevated temperatures because permanent chemical and physical changes occur within concrete heated to high temperatures [3]. This in turn reduces both the strength and the stiffness of the concrete, the parameters that are proportional to the ultrasonic pulse velocity measured.

Non-Destructive Tests Conducted

Throughout the course of the experiments conducted as part of this thesis, several of the columns failed during either the heating or the cooling phase of the experiments. As a result, residual experiments and non-destructive experiments were not conducted on these specimens. Only the concrete columns that survived beyond the cooling phase have been tested, as they remain structurally viable and cannot be effectively evaluated only by a visual inspection. In the case of a concrete column surviving the heating and cooling process, an ultrasonic pulse velocity test was conducted to produce a 'map' of the damage. This could then be compared to the actual measured temperature data within the concrete gathered using the embedded thermocouples during heating and cooling. The specimens tested and their corresponding heating/ loading conditions are given in Table 5.1.

Table 5.1: UPV Tests Conducted

Specimen identifier	Concrete Strength (MPa)	Load (%)	Load eccentricity (mm)	Heat Flux (kW/m^2)	Side(s) Heated
30-L60-E25-HF50-C-HL33	30	60	25	50	Compression
30-L60-E5-HF50-C-HL33	30	60	5	50	Compression
30-L60-E25-HF70-C-HL33	30	60	5	70	Compression
30-L1-E5-HF70-C-HL33	30	1	5	70	Compression
30-L1-E5-HF70-T-HL33	30	1	5	70	Tension
50-L60-E5-HF70-T-HL33	50	60	5	70	Tension
50-L1-E5-HF70-C-HL33	50	1	5	70	Compression
50-L1-E25-HF70-C-HL33	50	1	25	70	Compression
50-L1-E25-HF70-CT-HL33	50	1	25	70	Both
50-L60-E5-HF70-CT-HL33	50	60	5	70	Both

5.2.3 Mapping Damage

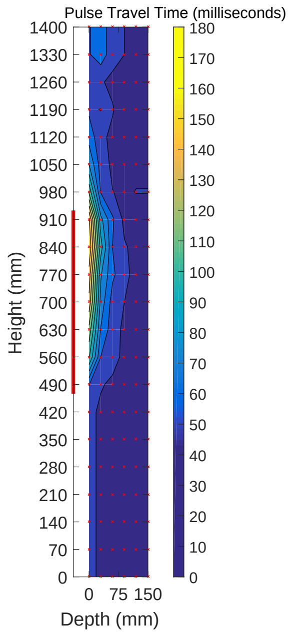
In mapping the damage resulting from exposure to elevated temperatures, as described in Chapter 3, only two values for the heat flux were investigated as part of this experimental tests series. It is also clear from Section 4.3 that the experiments conducted at similar heat fluxes experienced similar thermal gradients throughout the concrete sections. This is with the exception of columns that deflected in opposing directions (toward or away from the heat source). However, as discussed in Section 4.3, this is of minor consequence when looking at the structural response due to the fact that concrete provides little structural benefit above around 500^{o}C [28]. Despite the fact that the temperatures at the surface of the columns vary by around 100^{o}C, as the difference in heat flux is relatively small, the temperature gradient deeper within the sections (where the concrete remains structurally viable) are more consistent. Therefore, it would be expected that, when carrying out any non-destructive test, that the temperature gradient predicted by the method would be reasonably consistent throughout all of the experiments conducted as part of this investigation, with potentially varied predictions for the damage close to the surface of the columns. See Section 4.3 for a discussion on the measured temperature evolution through the columns.

When measuring the ultrasonic pulse velocity of the specimens before and after exposure to elevated temperature, measurements were taken in the same location for all of the specimens. All measurements were taken in a grid pattern 30mm by 70mm. This is with the exception of the first UPV test conducted on column No.3, where a grid of 50mm by 70mm was used. The grid pattern was altered for the remaining experiments as a result of the resource required to measure with such resolution. In order to create a heat map of the ultrasonic pulse velocity of the specimens, the pulse velocities were linearly interpolated between measurements.

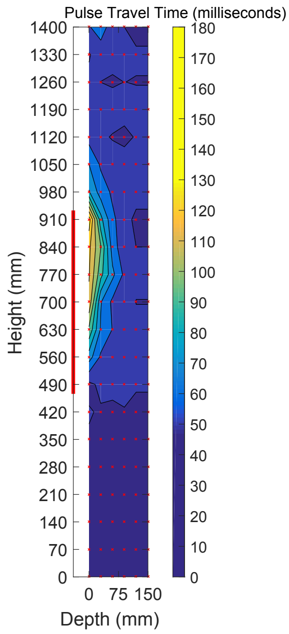
After conducting the UPV tests detailed in Table 5.1, the heat maps shown in Figures 5.1 and 5.2 detail the pulse velocities calculated through the section.

C30 Concrete Columns

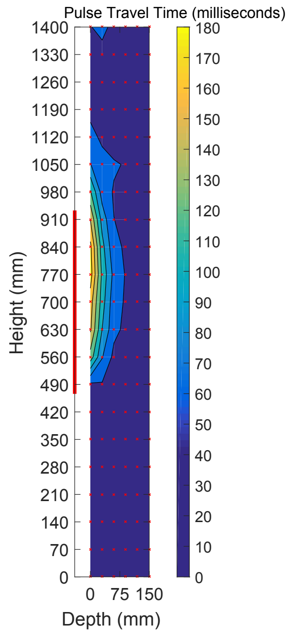
All of the following heat maps detailed in Figure 5.1 are for C30 reinforced concrete columns exposed to 50kW/m^2 or 70kW/m^2 for a total of 90 minutes. Measurements were taken side-to-side to visualise the temperature gradient through the column. The columns were then allowed to cool for a total of 24 hours before the UPV measurements were taken.



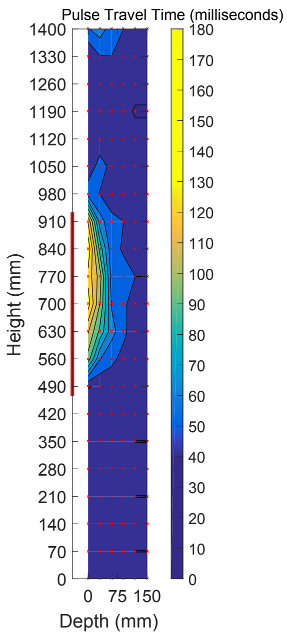
(a) 30-L60-E25-HF50-C-HL33



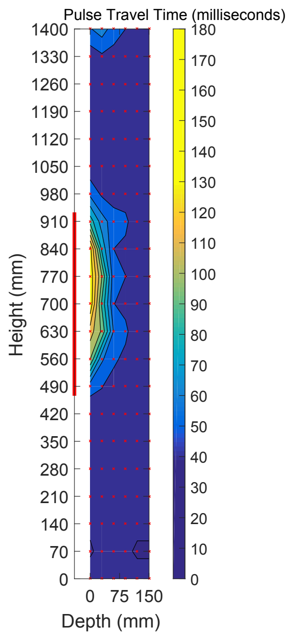
(b) 30-L60-E5-HF50-C-HL33



(c) 30-L60-E25-HF70-C-HL33



(d) 30-L1-E5-HF70-C-HL33

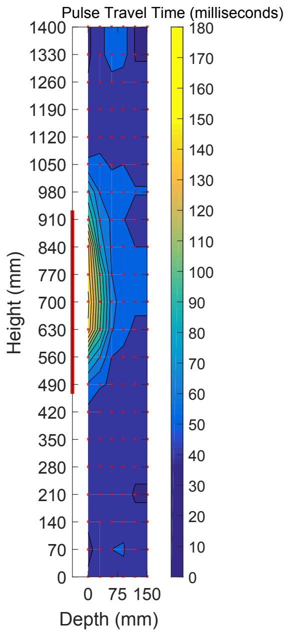


(e) 30-L1-E5-HF70-T-HL33

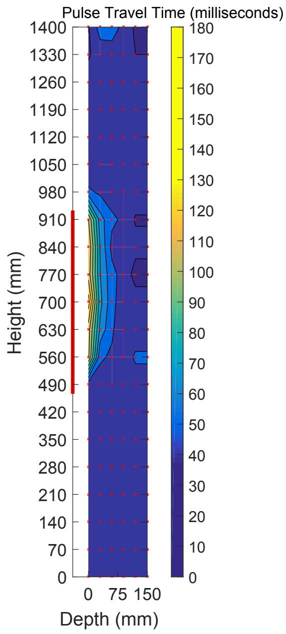
Figure 5.1: Pundit measurements taken on C30 concrete columns surviving the heating and cooling phase of the experiment subjected to a 50kW/m^2 or 70kW/m^2 incident heat flux

C50 Concrete Columns

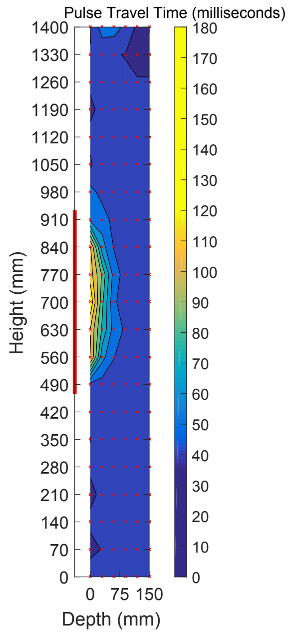
All of the following heat maps detailed are for C50 reinforced concrete columns exposed to a 70kW/m^2 for a total of 90 minutes. Measurements were taken side-to-side to visualise the temperature gradient through the column. The columns were then allowed to cool for a total of 24 hours before the UPV measurements were taken.



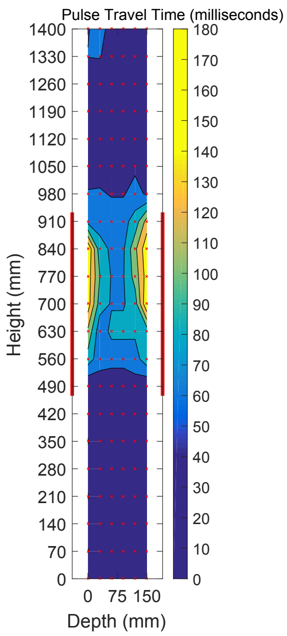
(a) 50-L60-E5-HF70-T-HL33



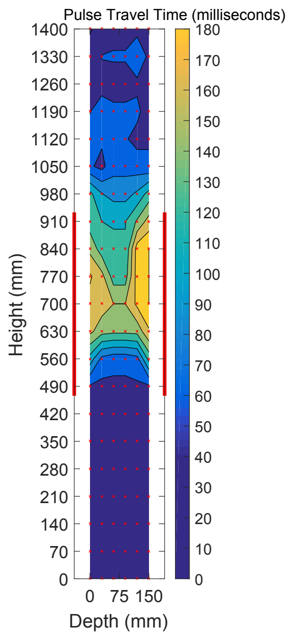
(b) 50-L1-E5-HF70-C-HL33



(c) 50-L1-E25-HF70-C-HL33



(d) 50-L1-E25-HF70-CT-HL33



(e) 50-L60-E5-HF70-CT-HL33

Figure 5.2: Pundit measurements taken on C50 concrete columns surviving the heating and cooling phase of the experiment subjected to a 70kW/m^2 incident heat flux

Discussion

It is quite clear from the pulse velocity maps that heating the columns in the middle third has a marked effect on the pulse velocities through the section after cooling to ambient temperature conditions. Pulse velocities throughout the columns were measured before the heating for all columns and were consistent with the ‘undamaged’ areas of the columns detailed in Figures 5.1 and 5.2 over the entire length of the columns. This is in contrast to the pulse velocities measured after being exposed to elevated temperatures where, in the area of damage, the pulse velocities measured were observed to have slowed drastically. It is therefore clear that there is a correlation between the damage inflicted on the column and the pulse velocities through the sections.

As can be seen in all of the maps detailed in Figure 5.1 and Figure 5.2, the area of the slowest pulse velocity is consistently located at the heated surface, slightly above the centre of the column and, in addition to this, the area of damage at the surface of the columns extends further above the mid-height of the column than it does below the centre. This is not something that would be expected if the only mode of heat transfer is radiation, which would be expected to damage the column in a symmetrical manner equally above and below the radiant panel in the heating condition used. Looking at the radiant panels and modes of heat transfer in isolation, there are credible explanations as to why this occurs. The reasons behind this have been described in more detail in Section 3.3.3.

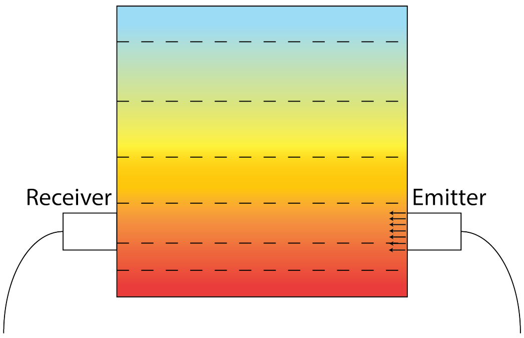
5.2.4 Estimating Temperature

The next step in determining the “damage” to the concrete columns is to relate the UPV readings to the relative strength of the concrete. It has previously been demonstrated in a number of studies [74, 75, 77, 88, 92] that the ultrasonic pulse velocity of the concrete changes as the concrete degrades at, and after exposure to, elevated temperature. As a result of this it may be possible to determine, for a specific concrete mix, an empirical relationship between the highest temperature achieved by the concrete and the ultrasonic pulse velocity reading. Note that any correlation between the concrete mix and the relationship between the ultrasonic pulse velocity and the maximum temperature achieved during the heating phase will strictly only be relevant to the concrete mix used as part of this experimental test series only.

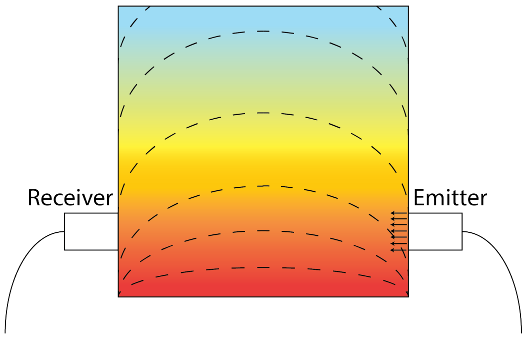
To determine such an empirical relationship, ideally a number of concrete cylinders would be cast using the same concrete mix used for the structural elements. These specimens would then be heated to a uniform temperature and allowed to cool to ambient temperature conditions for 24 hours before having their pulse velocities tested. As this was not possible as part of this experimental test series, the empirical relationship has been determined from the experiments on the concrete columns themselves. An estimation of the pulse velocity of the concrete at a range of temperatures could be compared against the temperatures measured in the concrete columns via the embedded thermocouples, thus enabling an empirical relationship to be determined between the pulse velocity and the maximum temperature achieved in the concrete.

This approach is in addition not possible in practice as any building that experiences a fire is very unlikely to have concrete cores of the exact concrete mix, stored in the same conditions for the same period of time, available to be used for such an investigation. The other approach in practice would be to take cores of undamaged material from the elements to allow them to be tested in the lab. The possible challenges of this approach include the fact that the cores taken will not contain thermocouples that have been cast into the concrete to measure the temperature within. The concrete cores would therefore have to be heated to the desired temperature with no internal instrumentation, resulting in uncertainty as to what temperature the centre of the concrete core actually achieved. As a result of this uncertainty and due to the fact that the purpose of measuring the pulse velocity of the specimens is an initial scoping exercise to determine if this technique has potential applications, no cores were tested in estimating the correlation between the pulse velocity and the maximum temperature achieved.

As thermocouples were cast into the reinforced concrete columns themselves it is possible to estimate this correlation for each concrete mix using the columns themselves. This correlation can then be applied to other reinforced concrete columns tested to determine if this method can potentially be applied in practice. Note that it is possible to estimate the pulse velocity of the concrete at certain temperatures due to the fact that the columns were heated on one side only. Therefore one dimensional heat transfer has been assumed through the section. This however is not the case in reality. As the heat transfer through the concrete section is two dimensional in nature, heat will be lost at the sides of the sections, resulting in a non-uniform temperature across the width of the element. The temperature measured by the thermocouples is in the centre of the section. Due to the fact that the ultrasonic pulse will travel more slowly through concrete that achieved a higher temperature, this technique will overestimate the speed of the ultrasonic pulse for each temperature. This has been detailed in Figure 5.3.



(a) Assumed temperature isotherms (1D heat transfer)



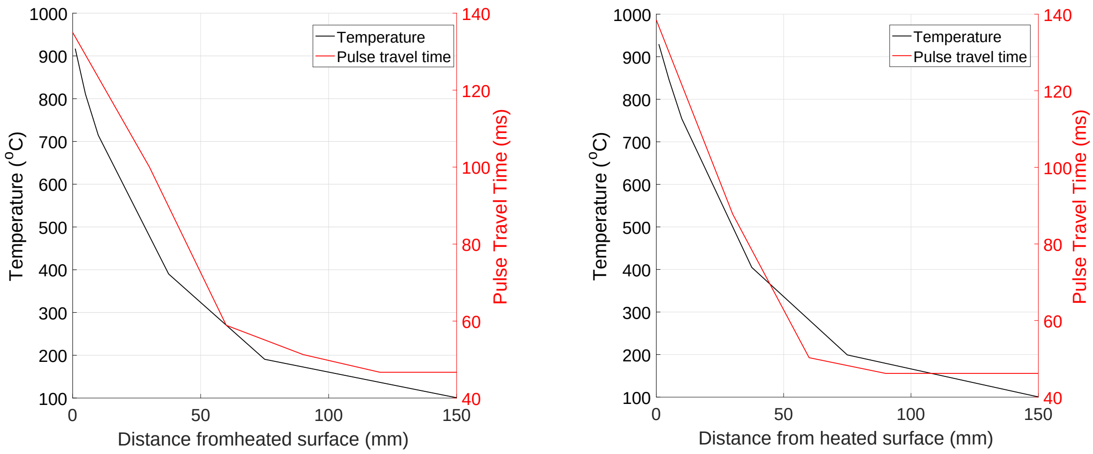
(b) Actual temperature isotherms (2D heat transfer)

Figure 5.3: Illustration of the assumed temperature isotherms when calculating the relationship between pulse velocity and concrete strength vs the actual isotherms in the concrete

Section 5.2.5 details the correlation between the ultrasonic pulse velocity measured and the maximum temperature measured within the specimens at a given depth.

5.2.5 Pulse Velocity - Temperature Correlation

As detailed in Section 5.2.4, the correlation between the ultrasonic pulse velocity and the maximum temperature achieved was estimated using the thermocouples in the reinforced concrete columns cast. Figure 5.4 shows the temperature gradient through the specimens and the ultrasonic pulse velocities measured across the width of the specimens. Note that the temperatures displayed in Figure 5.4 do not display the temperature gradient through the specimen at any given time during the experiment. Rather, it illustrates the maximum temperature measured by each thermocouple during the experiment (which will not necessarily occur at the same moment in time due to the fact that, when the heat source is switched off, the thermal wave will continue to travel through the concrete, increasing the temperature at the back of the specimen despite the column officially entering the cooling phase of the experiment).



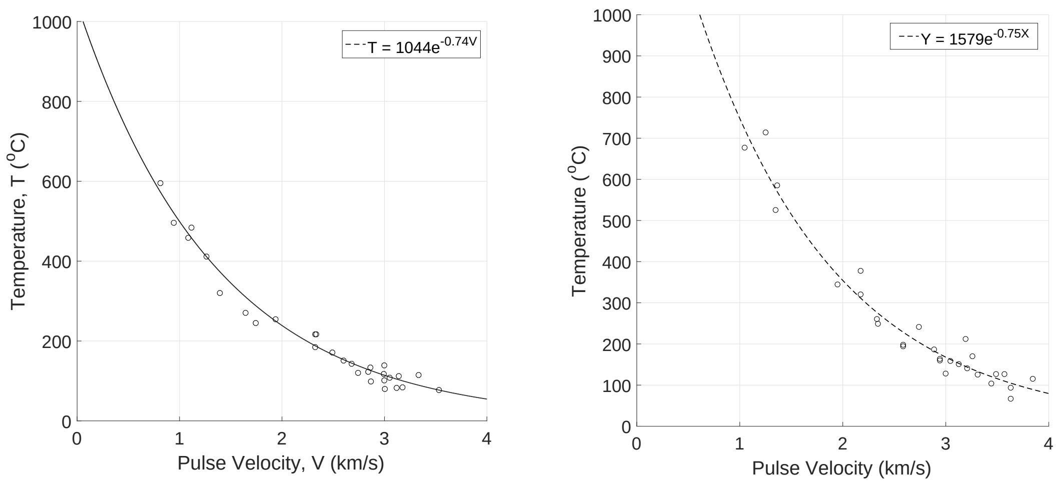
(a) 30-L1-E5-HF70-C-HL33 **(b)** 30-L1-E5-HF70-T-HL33

Figure 5.4: Simultaneous plots of the maximum experienced temperature and post-cooling pulse velocity measurements taken at prescribed distances from the heated surface

From Figure 5.4, the maximum temperature in the specimens throughout the experiments conducted is located at the heated surface and gradually decreases to a minimum on the other side of the section. Compare this to the ultrasonic pulse velocities measured at each point across the width of the specimens and a clear correlation is evident. As the maximum temperature in the concrete decreases, the time taken for the pulse to travel through the column also decreases.

From the pulse velocities and temperatures detailed in Figure 5.4, a correlation has been created to link the two measurements. It is envisaged that this correlation may be used to identify the maximum temperature at any point through the column and, ultimately, build a profile of the damage to the column in real terms i.e. strength, elasticity etc.

To relate the pulse velocity with the maximum temperature achieved during the experiments, all of the data collected for all of the reinforced columns of each strength have been plotted on one graph. In order to relate the two parameters, a least squares line of best fit has been calculated and plotted on the graph. This can then be used to determine the maximum temperature achieved at any point of the columns after cooling back to ambient temperature.



(a) C30 concrete columns

(b) C50 concrete columns

Figure 5.5: Scatter plot of all data collected from thermocouples and pulse velocity readings

Note that the correlations detailed in Figure 5.5 cannot be used to relate the pulse velocity with the maximum temperature for all types of concrete or all elements. As every concrete mix is composed of different materials and mix proportions, the properties of concrete may differ to a relatively large extent. Therefore, the correlations detailed in Figure 5.5 may only be used for the analysis of the concrete cast as part of this experimental test series. All other concrete mixes must be analysed separately to determine a similar relationship between the pulse velocity and the maximum temperature achieved after cooling.

5.2.6 Discussion of Results

Section 5.2 has detailed the results of the non-destructive experiments completed on ten of the reinforced concrete columns that survived the heating and cooling stages of the experiments. This has resulted in correlations for the relationship between the pulse velocity readings measured and the maximum temperatures experienced by the concrete during the experiments detailed in Figure 5.5.

Uncertainties

There are a number of limitations to the experimental procedure followed in the construction of the correlations in Figure 5.5. The location of the thermocouples used to measure temperature, despite being tightly controlled, may have moved slightly (≈1-2mm) during casting and curing of the concrete. However, as the movement of thermocouples was restricted, it is envisaged that the error introduced by this would be small. The Pundit used in measuring the pulse velocity consisted of a hand-held emitter and receiver with a total surface diameter of 30mm. A smooth contact between the emitter, receiver, and the concrete was ensured with plasticine between the interfaces. This however created an interface which accumulated dirt and other particles and, depending on the pressure imposed, may have changed shape and thickness from one measurement to another. In addition to this, the measurements taken would fluctuate, sometimes drastically, depending on the pressure imposed by the user. These fluctuations may result in errors being introduced, especially when the measurements are taken by different people applying different levels of pressure. The fluctuations in the readings taken were also a result of the fact that the Pundit used to measure the pulse velocities was manufactured almost 40 years previously. In this time, the consistency and accuracy of such devices has improved with advances in technology. Therefore, to reduce the errors with regard to this aspect of the experiments, it is possible to use a more modern version of the Pundit.

The largest uncertainty in the production of the correlations detailed in Figure 5.5 would however come from the linear interpolation of the temperature data between thermocouples. Figure 5.4 details the temperature gradient through two of the columns compared to the pulse velocity measurements taken. It is clear from this figure that the thermocouples are not located in the same location as the pulse velocity measurements taken. In order to correlate these values, the temperature at the point of the pulse velocity measurements was assumed based on linear interpolation of the two nearest thermocouples. It is however clear that interpolating the data in this manner will overestimate the temperature in the concrete at this point.

Despite the uncertainties detailed, the process of predicting the maximum temperature of the concrete based on pulse velocity measurements holds merit. The next steps in increasing confidence in this method is to increase the volume of data available for comparison with real thermocouple data. It is necessary to complete additional experimental test series designed specifically to measure these values taking account of the uncertainties described.

Applicability and Limitations

Taking note of the positive results outlined in Section 5.2, it is clear that there is potential for the use of the UPV method in estimating the extent of damage inflicted to concrete by exposure to elevated temperatures. It should, however, be noted that the conditions of the experiments conducted were almost "ideal" for the application of such a method. The columns tested were constructed, conditioned, and tested in a controlled environment. When exposed to elevated temperature, the exact location and heat flux were tightly controlled to be as one dimensional as possible. In addition to this, the sides of the columns were easily accessible after the experiment for measurements to be taken. It should be acknowledged that, in the event of a fire within a building, the conditions of the fire may pose additional challenges. As the output of the measurements taken is that of a single number (the average speed of the pulse), the temperature of the concrete through which the pulse is travelling should ideally be uniform. However, with exposure on multiple sides, this is unlikely to be the case. This has been detailed in Figure 5.3. In addition, without access to the sides of the columns, it would not have been possible to determine the damage profiles detailed in Section 5.2.3. Measurements taken from "front to back" of the columns are considerably less valuable in determining the damage profile because measurements were taken parallel to the direction of travel of the temperature gradient. This resulted in a pulse velocity that passed through concrete that was both at 100^{o}C at the back and 1000^{o}C at the front. It was therefore not possible to determine the condition of the concrete at that location due to this variation.

Under certain conditions however, it is conceivable that the application of this method could provide engineers with an additional tool to be used in predicting the residual damage sustained by a concrete element as a result of exposure to elevated temperatures.

5.3 Destructive Tests - Method

Much like the heating/cooling phases described in Chapter 4, the results of the residual strength experiments are to be provided on a case by case basis on how each parameter may alter the structural response when investigated in conjunction with the other parameters. As many of the specimens failed during the heating or cooling phase of the experiments, it was not possible to conduct residual strength experiments on all of the specimens. Any specimen that survived until the end of the cooling phase was tested to failure 24 hours after being exposed to elevated temperatures.

5.3.1 Experimental Method

As described in Chapter 3, the residual strength experiments were conducted 24 hours after the columns were initially exposed to elevated temperatures. The columns were subjected to a linear load ramp of 15kN/min until failure. The load was applied at the same load eccentricity as was applied to the columns 24 hours previously.

As can be seen in Chapter 4, after cooling back to ambient temperature and having the load removed, the columns did not necessarily return to ambient temperature conditions regarding the midspan deflections. Residual deflections remained in the columns. Therefore, to determine the residual performance of the reinforced concrete columns, it is not sufficient to simply consider the loss in strength and stiffness of the columns as a result of being exposed to elevate temperatures. The residual midspan deflections will effectively increase or decrease the eccentricity of the load applied, which may ultimately be to the benefit or detriment of the columns load carrying capacity. Where residual displacements were present, details have been provided.

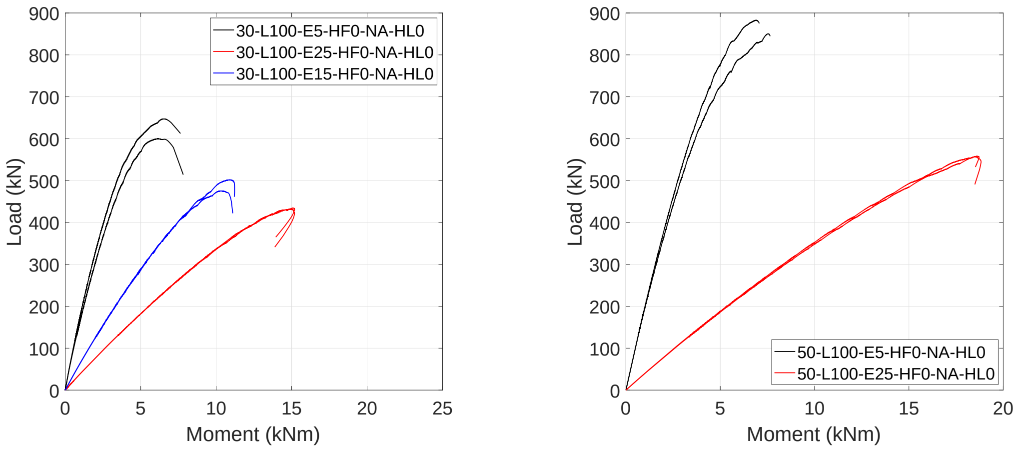
5.3.2 Ambient Experiments

As discussed in Section 4.2, a series of ambient temperature experiments were conducted on the columns before the heated experiments on order to determine their ambient temperature response. The ambient temperature response of the reinforced concrete columns to the load conditions investigated have been summarised in Table 5.2. For full details of the ambient temperature experiments conducted, see Section 4.2.

Table 5.2: Ambient Tests Conducted

Concrete Strength (MPa)	Load eccentricity (mm)	Ultimate Load (kN)
30	5	647.21
	5	599.77
	15	501.72
	15	475.07
	25	431.90
	25	434.35
50	5	850.31
	5	882.63
	25	558.01
	25	557.08

In order to fully understand the response of the columns and compare them to the ambient temperature experiments conducted, it is not possible to simply compare the load-deflection responses. This is a result of the residual deflections within the damaged columns resulting in an increased (or decreased) bending moment. Therefore, in addition to the load deflection of the columns, the load-moment (M-N) interaction diagrams are plotted for the load path taken in each experiment. Figure 5.6 details the M-N diagrams for each of the ambient temperature experiments. This has been compared to both the residual strength experiments in Section 5.4 and to the expected results from the sectional analysis model created in Chapter 6.



(a) C30 concrete columns

(b) C50 concrete columns

Figure 5.6: Ambient load path (load-moment) diagrams for (a) C30 and (b) C50 concrete columns at midspan

As is expected, higher load eccentricities increase the moment within the columns which, as a result, fail at a smaller value of axial load due to the increased stress state at the extreme fibre of the section. It would be expected that the reinforced concrete columns follow an envelope which defines failure of a specific reinforced concrete section. For full details of this calculation and a comparison of the failure envelope to the ambient temperature experiments conducted, see Chapter 6.

The graphs displayed in Figure 5.6 have been modified to account for the secondary moments induced in the concrete columns as a result of the increased deflections as the load increased. The moment in the column has been calculated for each time step in the experiment using the Equations 5.2a, 5.2b, and 5.2c.

$$M_1 = \varepsilon \times N \tag{5.2a}$$

$$M_2 = \delta \times N \tag{5.2b}$$

$$M = M_1 + M_2 \tag{5.2c}$$

Where:

M = The total moment induced in the column (kNm)

M_1 = The 1^{st} order moment induced in the column (kNm)

M_2 = The 2^{nd} order moment induced in the column (kNm)

N = The load applied to the column (kN)

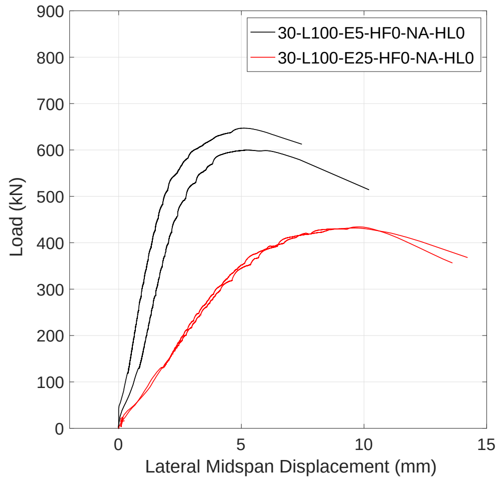
ε = The eccentricity of the load applied (m)

δ = The lateral midspan deflection of the column (m)

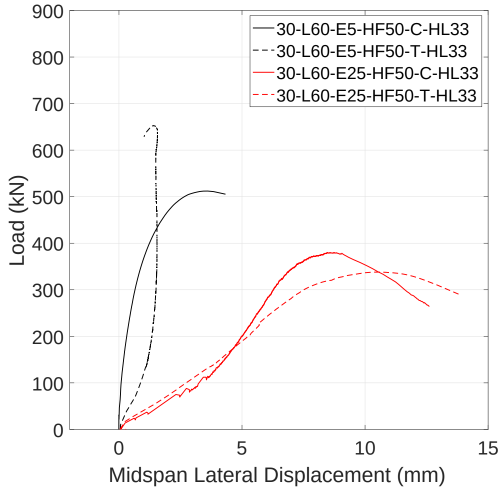
5.4 Residual Experiments

5.4.1 C30 Concrete Columns - 50kW/m^2 Heat Flux

The following Figure details the results of the post-cooling residual strength experiments conducted on the weaker, C30, reinforced concrete columns exposed to a heat flux of $50kW/m^2$ during the heating phase of the experiment in comparison to the ambient temperature experiments conducted on effectively identical concrete columns.



(a) Ambient results



(b) Residual results

Figure 5.7: Load-displacement for post-cooling residual experiments on C30 columns subjected to a $50kW/m^2$ heat flux

Looking at Figure 5.7, it would seem that exposure to elevated temperature before cooling and being tested residually 24 hours later has had a marked effect on the response of the columns. Despite the fact that, as with the ambient temperature strength experiments, only two load eccentricities were applied (5mm and 25mm), it is clear that the repeatability achieved during the ambient temperature strength experiments has not been achieved after heating and cooling. The 25mm load eccentricity experiments display relatively similar behaviour in that they are clearly weakened by the damage induced in the column 24 hours previous. This is in comparison to the 5mm load eccentricity experiments which behave in a different fashion. The "compression heated" column behaves as expected, as the load increases, the midspan deflection increases until the column fails at a smaller load than the ambient temperature experiments due to the weakening of the concrete. However, the "tension heated" specimen appears to get stiffer as the load is increased and fails at a slightly higher load than the ambient temperature experiments. This behaviour is unexpected due to the fact that the column has been damaged.

However, as discussed a number of times in Chapter 4, the reinforced concrete columns deflect during the heating phase. During the loaded experiments conducted, this deflection was not fully recovered after cooling when the axial load was removed. Of the parameters investigated in this experimental test series, the extent and direction of the residual deflection in the columns is primarily a function of the side(s) heated and the eccentricity of the load. For a full discussion on the residual deflections in the columns see Chapter 4. Table 5.3 details the maximum load achieved in the residual strength experiments, the residual midspan displacement of the column after being exposed to elevated temperatures, and the effective load eccentricity as a result after fully cooling back to ambient temperature conditions.

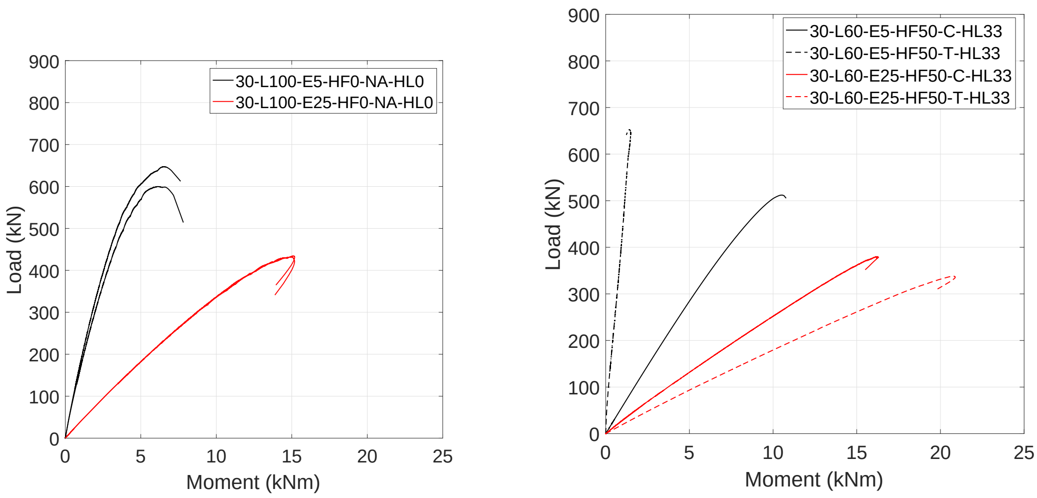
Table 5.3: Residual deflection of C30 concrete columns subjected to a $50kW/m^2$ heat flux

Specimen identifier	Load eccentricity (mm)	Residual deflection (mm)	Effective load eccentricity (mm)
30-L60-E5-HF50-C-HL33	5	12.0	17
30-L60-E25-HF50-C-HL33	25	14.1	39.1
30-L60-E5-HF50-T-HL33	5	-5.8	-0.8
30-L60-E25-HF50-T-HL33	25	9.0	34.0

Therefore, due to the permanent bowing of the concrete columns as a result of being exposed to elevated temperatures, the initial eccentricity of the load will be increased or decreased depending on the direction the column bowed during the heating and cooling phases.

The residual displacement of the 5mm load eccentricity, "tension heated" experiment was -5.8mm (reducing the total eccentricity of the load). The initial load eccentricity for this experiment was 5mm. Therefore, due to the residual displacement of the column after cooling, the load applied during the residual strength experiment would be at an eccentricity of roughly -0.8mm (almost concentric). As the load eccentricity of the residual experiment was smaller than the eccentricity of the ambient temperature experiment, this may explain why, despite being damaged, the column achieved a higher ultimate load than that of the ambient temperature experiment conducted. As a result of the smaller moment in the column, it could resist higher axial loads despite having been damaged.

Taking this into account it is possible to plot the M-N load paths for the reinforced columns exposed to the lower heat flux using the method described in Section 5.3.2.



(a) Ambient experiments

(b) Residual experiments on heated columns

Figure 5.8: Residual M-N load paths for C30 concrete columns subjected to a 50kW/m^2 at 60% ultimate axial load during heating

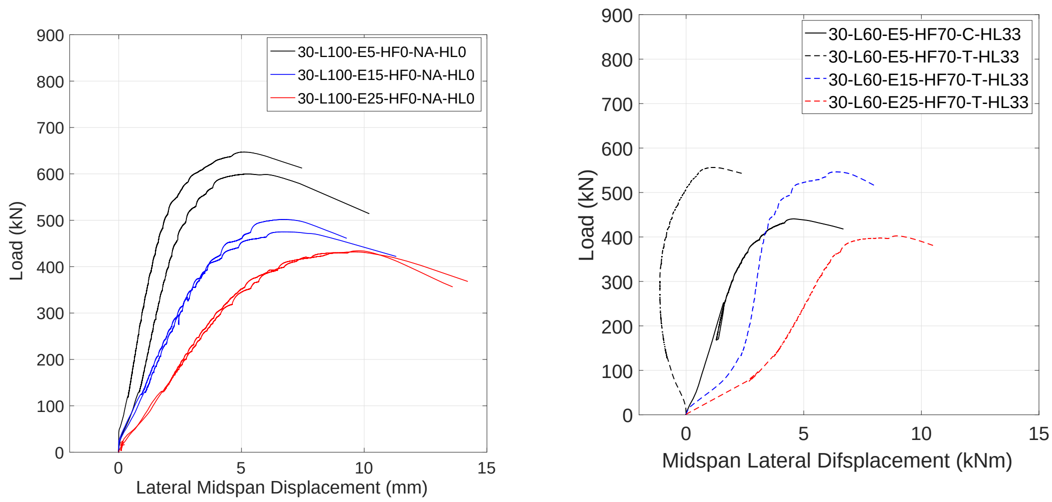
Figure 5.8 illustrates the load paths taken during each of the experiments taking both the initial residual deflection, and the load induced deflection into account. It can be seen that, when plotted against the actual moment induced in the column, the experiments follow a more predictable load path. As the residual displacements of the columns determine the initial moment within the column, the plots do not follow the same paths as for the ambient temperature experiments. However, as discussed in Section 5.3.2, every section has a M-N failure envelope that it is expected to follow. The residual experiments conducted on the reinforced concrete columns appear to fail in a manner that seems to follow this pattern. For full details of the failure envelope expected from the reinforced concrete columns and a comparison with the data collected from the experiments, see Chapter 6.

Therefore, despite the residual behaviour of the columns being inconsistent in terms of the load path taken, the response of the columns is as expected when taking the initial residual midspan deflection into account. However, in real world terms, this may a difficult task. To the naked eye, the residual displacement of the columns is small and difficult to even perceive in some cases. In addition to this, depending on the side heated and the load eccentricity, the column could deflect in both directions which could either increase or decrease the ultimate axial load of the element under that specific structural arrangement. As the direction and magnitude of the residual deflection has such a large effect on the residual performance of the reinforced concrete columns (see Figure 5.8b), the load path taken by a potential in-situ column after cooling back to ambient temperature is difficult to determine.

This raises a number of questions when it comes to determining the ability of an element to withstand a certain load condition after a fire. For a full discussion of these questions and possible ways forward, see Section 5.5.1.

5.4.2 C30 Concrete Columns - 70kW/m^2 Heat Flux

The following figures detail the results of the residual strength experiments conducted on the weaker, C30, reinforced concrete columns exposed to a heat flux of $70kW/m^2$ during the heating phase of the experiment. Due to the fact that the experiments conducted on the reinforced concrete columns subjected to a 25mm load eccentricity in a "compression face" heating scenario failed during the cooling phase of the experiment, no residual experiment was conducted for these experiments.



(a) Ambient experiments

(b) Residual experiments on heated columns

Figure 5.9: Residual load-displacement for C30 concrete columns subjected to a 70kW/m^2 at 60% ultimate axial load during heating

From Figure 5.9, it is clear that the columns again display very different behaviour to that of the ambient temperature experiments. The experiment conducted on the column subjected to elevated temperatures on the "tension face" with a 5mm load eccentricity displays an especially strange behaviour. Immediately upon loading, this column deflects in one direction and then, between a load of 100kN and 300kN, appears to get stiffer. The column then transitions and begins to deflect in the opposite direction until failure. This behaviour may yet again be explained when taking the residual displacements of the column after cooling into account. After heating and cooling, this column sustained a residual deflection of -10.9mm. As the initial load eccentricity was only 5mm, this column transitioned from being "tension heated" to being "compression heated" during the heating and cooling phases of the experiment. Therefore, the actual load eccentricity applied during the residual strength experiment would have started at -4.9mm. Upon first loading the column, it settles as a result of the small load eccentricity. When the load is increased beyond 300kN, as the load eccentricity is negative, the column begins to deflect, and ultimately fail in the opposite direction to what would be expected.

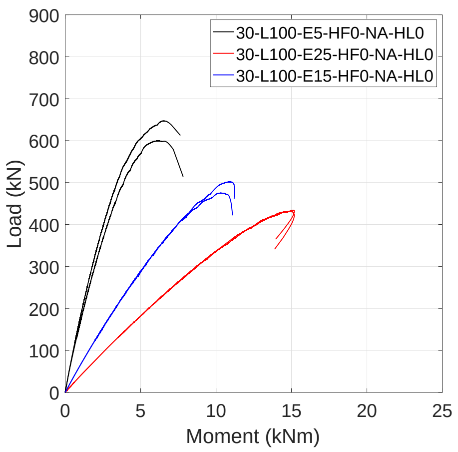
In addition to this, all of the "tension heated" specimens, despite not displaying the same behaviour in terms of the load path, all achieve a comparable ultimate load with the ambient temperature experiments conducted on the columns in the same load condition before heating. However, the "compression heated" specimens either failed during the heating/cooling phases or had a considerable reduction in strength when it came to their residual response.

With regard to the "tension heated" columns that did not transition into being compression heated during the course of the experiment, it is clear that they experience higher deflection upon first loading before appearing to get stiffer as the load is applied further before failing at roughly the same location as the ambient temperature experiments. For a full discussion of this behaviour, see Section 5.4.8 where a similar, but more pronounced behaviour occurred with the columns subject to an increased heated length on the "tension face".

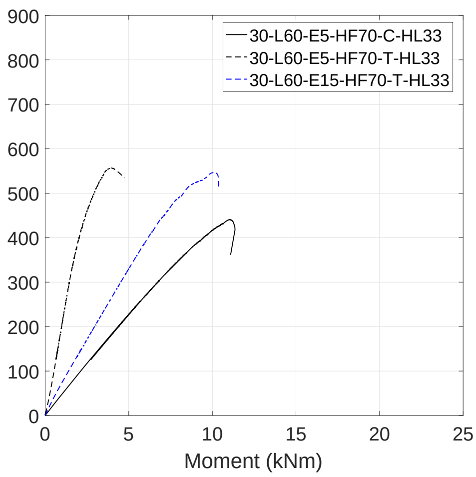
As with the columns subjected to a low heat flux, the M-N load paths for the columns have been plotted in Figure 5.10 with details of the residual displacement provided in Table 5.4.

Table 5.4: Residual deflection of C30 concrete columns subjected to a $70kW/m^2$ heat flux

Specimen identifier	Load eccentricity (mm)	Residual deflection (mm)	Effective load eccentricity (mm)
30-L60-E5-HF70-C-HL33	5	15.5	20.5
30-L60-E25-HF70-C-HL33-(1)	25	Failed	Failed
30-L60-E25-HF70-C-HL33-(2)	25	Failed	Failed
30-L60-E5-HF70-T-HL33	5	-10.9	-5.9
30-L60-E15-HF70-T-HL33	15	-2.9	12.1
30-L60-E25-HF70-T-HL33	25	Unknown	Unknown



(a) Ambient experiments



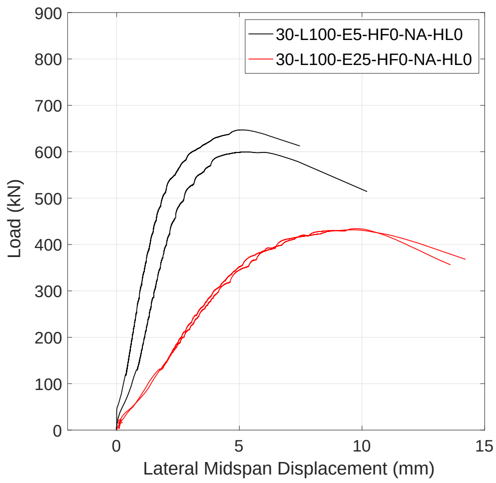
(b) Residual experiments on heated columns

Figure 5.10: Residual M-N load paths for C30 concrete columns subjected to a 70kW/m^2 at 60% ultimate axial load during heating

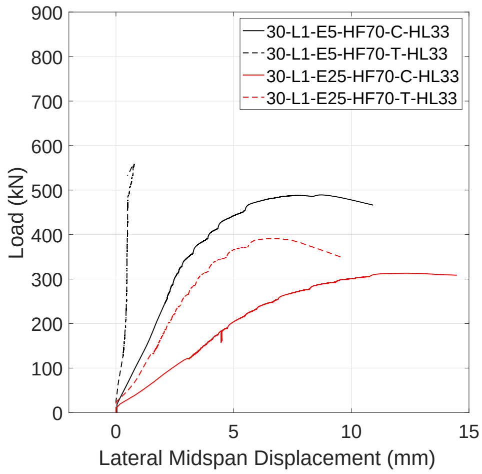
From Figure 5.9 it was concluded that the "tension heated" specimens all failed at a comparable ultimate strength as the ambient temperature experiments conducted on the columns under the same loading condition. For the "tension heated" experiment with a 5mm load eccentricity, as would be expected from a column damaged by fire, the ultimate moment achieved at failure is smaller than for the ambient temperature experiments. This is, however, not true for the experiment conducted on the "tension heated" column with a 15mm load eccentricity. This column achieves a higher ultimate load, with an almost identical ultimate moment at failure compared to the ambient temperature experiments conducted under the same loading conditions. This may be due to having a reduced effective load eccentricity during the residual experiment (12.1mm), and due to the inherent variability of concrete in general. See Chapter 6 for a detailed comparison of the experimental results with the sectional analysis model produced.

5.4.3 C30 Concrete Columns - 10kN Load

The following figures detail the results of the residual strength experiments conducted on the weaker, C30, reinforced concrete columns exposed to a heat flux of $70kW/m^2$ while subjected to a nominal axial load (10kN).



(a) Ambient experiments



(b) Residual experiments on heated columns

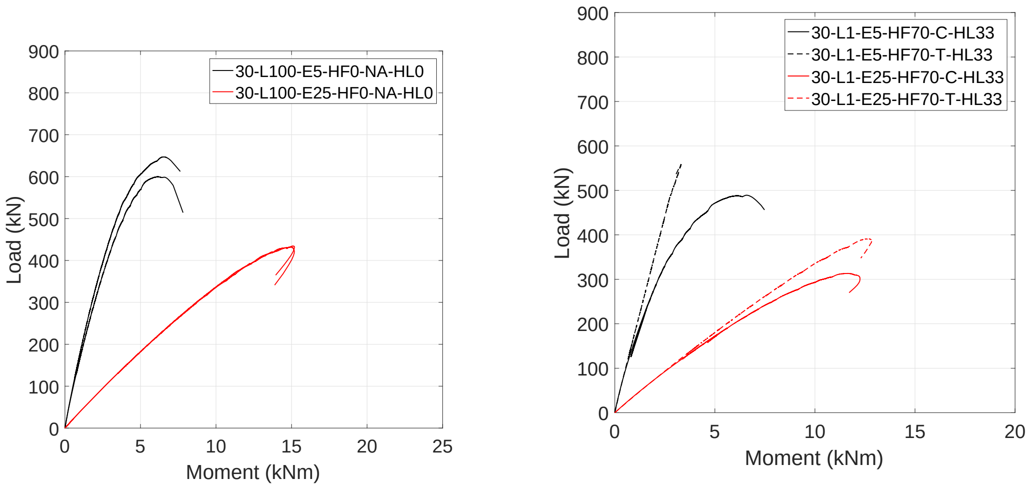
Figure 5.11: Residual load-displacement for C30 concrete columns subjected to a 70kW/m^2 and a 10kN axial force during heating

From Figure 5.11, similar to the columns tested under the higher (60%) load condition, the "tension heated" specimens tend to perform better when tested residually. Every experiment graphed in Figure 5.11 is of a column subjected to the exact same 70kW/m^2 heat flux while also loaded with a total of 10kN (≈1-2% of the columns total ambient temperature capacity). The "tension heated" columns therefore achieve a higher ultimate axial load because the concrete on the most highly stressed side of the concrete, the "compression face", is not as damaged as in the "compression heated" scenario. The "tension heated" columns are therefore stiffer and can resist a higher load because the "compression face" of the concrete is stronger. See Section 4.6.1 for a full discussion on how the stress is distributed through the sections of the columns.

Table 5.5 and Figure 5.12 detail the residual deflections in the columns after cooling back to ambient temperature, and the M-N load paths taken in the residual experiments.

Table 5.5: Residual deflection of C30 concrete columns subjected to a $70kW/m^2$ heat flux while subjected to a 10kN axial load

Specimen identifier	Load eccentricity (mm)	Residual deflection (mm)	Effective load eccentricity (mm)
30-L1-E5-HF70-C-HL33	5	-0.2	4.8
30-L1-E25-HF70-C-HL33	25	-1.8	23.2
30-L1-E5-HF70-T-HL33	5	0.2	5.2
30-L1-E25-HF70-T-HL33	25	0.7	25.7



(a) Ambient experiments

(b) Residual experiments on heated columns

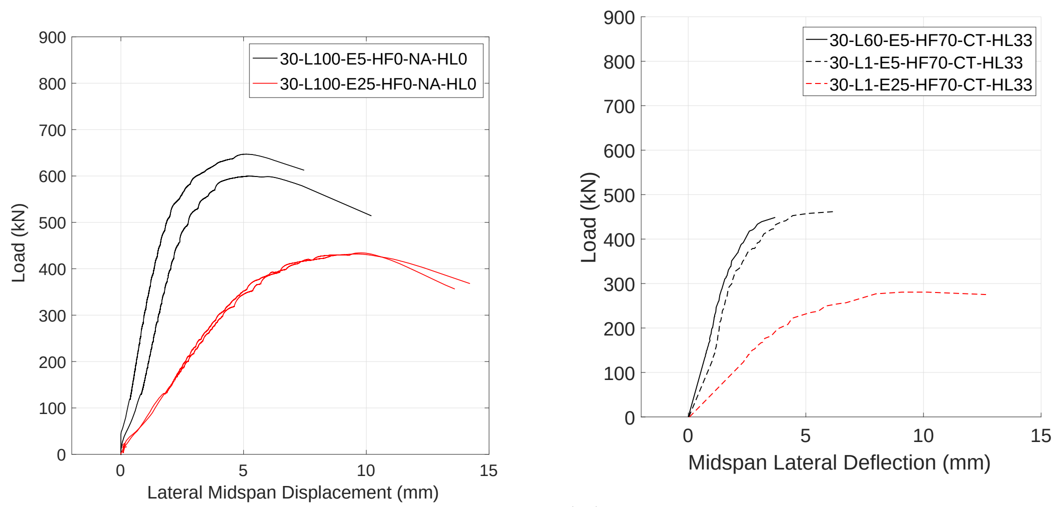
Figure 5.12: Residual M-N load paths for C30 concrete columns subjected to a $70kW/m^2$ at 60% ultimate axial load during heating

When considering the M-N load paths of the residual experiments on the columns subjected to a 10kN load during heating (Figure 5.12), it is clear that they do not follow the same pattern as that of the columns subjected 60% of their ambient temperature capacity during heating (Figure 5.10 and Figure 5.8). The general trend of the residual performance of the columns loaded to 60% of their ambient temperature capacity is that, residually, the ultimate load is reduced while the ultimate moment is increased for the "compression heated" specimens, whereas the "tension heated" specimens tended to behave in a similar fashion to the ambient temperature experiments, with a slight reduction in the axial force and moment at failure. The columns that were only partially loaded with 10kN during heating however see a reduction in both the ultimate axial load and the ultimate moment while following a similar load path to begin with for both the "tension heated" and the "compression heated" specimens.

It is well established that the strength of concrete decreases with increased temperature however, as this is the case, it would be expected that the loaded and unloaded columns would behave in a relatively similar manner (as they have been damaged to the same degree). There is therefore an additional mechanism taking place which results in the columns subjected to a 10kN load behaving in a more predictable manner than that of the columns loaded to 60% of their ambient temperature capacity. This may again be explained by considering the residual deflections of the columns. The residual displacements of the columns subject to a 10kN load during the initial phases of the experiments, as is detailed in Table 5.5, is almost negligible. Therefore, upon initial loading, the columns follow the same load path as the ambient temperature experiments. However, as the load is increased, the damaged columns divert and experience higher moments as a result of the damage sustained. The residual displacements of the columns subject to a 60% load during the initial phases of the experiments experience much higher residual displacements. This results in the damaged columns taking a completely different load path from initial loading of the residual experiments conducted.

5.4.4 C30 Concrete Columns - Double Sided Heating

The following figures detail the results of the residual strength experiments conducted on the weaker, C30, reinforced concrete columns exposed to a heat flux of $70kW/m^2$ on both the "tension face" and the "compression face" during the heating phase of the experiment.



(a) Ambient experiments

(b) Residual experiments on heated columns

Figure 5.13: Residual load-displacement for C30 concrete columns subjected to a 70kW/m^2 on both the "compression face" and the "tension face"

As would be expected from such an extreme heating regime, the ultimate load that can be held by the columns and their stiffness, in each of the structural arrangements, is greatly reduced. Unlike during the heating phases of the experiments, Figure 5.13b illustrates that there is no significant difference between the columns that were loaded to 60% of their ambient temperature capacity during heating and those that were subjected to a nominal load of 10kN during heating. When considering the columns heated on one side only, there is however a large difference in the load-deflection history of the columns subjected to differing load detailed in Sections 5.4.2 and 5.4.3. This is however a result of differing residual deflections sustained during the heating and cooling phases of the experiments. The larger residual displacements of the columns loaded to 60% of their ambient temperature capacity during heating resulted in higher moments during the residual experiments and ultimately, alternate load paths being taken to the columns loaded to 10kN during the heating phase. Whereas the residual displacements of the columns under both loading scenarios when heating on the "compression face" and the "tension face" were almost identical after cooling back to ambient temperature. This resulted in the columns subjected to both loading conditions taking the same initial load path and, ultimately, the same final load path due the same levels of damage sustained during heating.

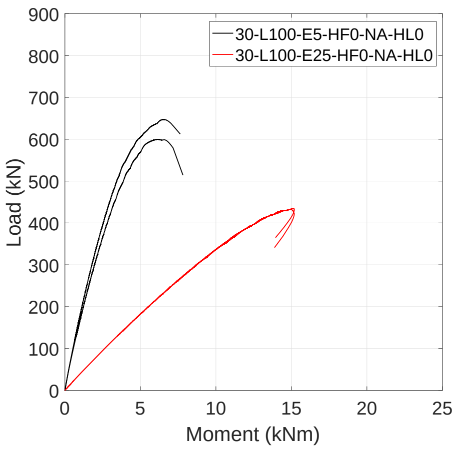
This result would suggest that the residual strength of the columns is dictated by the temperature/damage gradient through the column, and the load applied during the heating phase is inconsequential when considering residual strength only. However the load applied during heating will ultimately dictate the residual deflections of the element, and therefore the load path taken when loaded after cooling to ambient temperature conditions. Therefore, to effectively estimate the structural behaviour of the reinforced concrete element after being exposed to elevated temperatures, both the damage profile and the residual deflections are required.

This theory is interrogated further in Chapter 6, where a sectional analysis model has been developed to predict the residual strength of the concrete columns after being subjected to elevated temperatures.

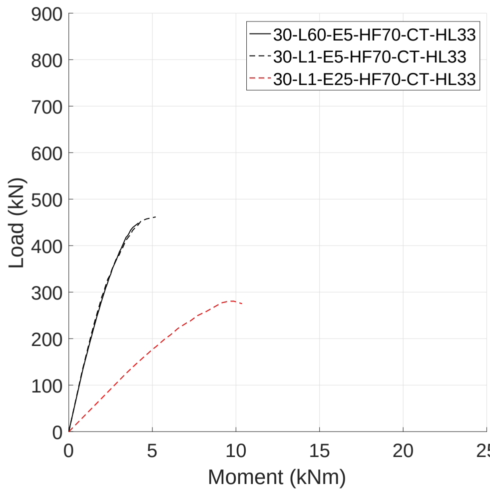
Table 5.6 and Figure 5.14 detail the residual deflections in the columns after cooling back to ambient temperature, and the M-N load paths taken in the residual experiments.

Table 5.6: Residual deflection of C30 concrete columns subjected to a $70kW/m^2$ heat flux on both the "tension face" and the "compression face"

Specimen identifier	Load eccentricity (mm)	Residual deflection (mm)	Effective load eccentricity (mm)
30-L60-E5-HF70-CT-HL33	5	0.6	5.6
30-L1-E5-HF70-CT-HL33	5	0.1	5.1
30-L60-E25-HF70-CT-HL33	25	Failed	Failed
30-L1-E25-HF70-CT-HL33	25	0.0	25



(a) Ambient experiments



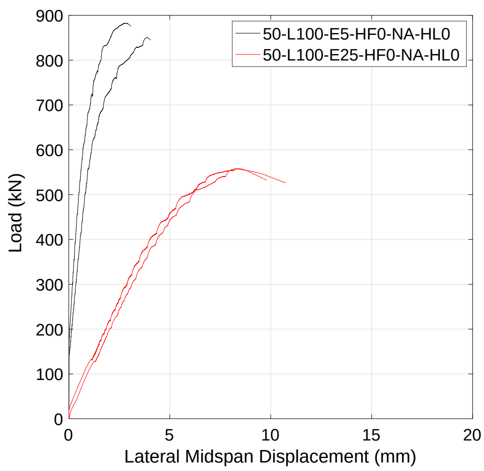
(b) Residual experiments on heated columns

Figure 5.14: Residual M-N load paths for C30 concrete columns subjected to a 70kW/m^2 on both the "compression face" and the "tension face"

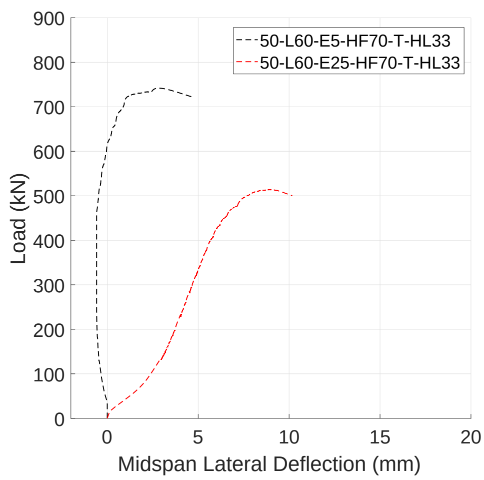
As has been discussed throughout this chapter with reference to the load-displacement of the columns, the capacity of all of the columns has been reduced as a result of exposure to elevated temperatures on both the "compression face" and the "tension face". In addition, the columns subjected to both heating conditions during heating have taken the same load paths in the residual destructive experiments conducted.

5.4.5 C50 Concrete Columns - 70kW/m^2 Heat Flux

The following figures detail the results of the residual strength experiments conducted on the stronger, C50, reinforced concrete columns exposed to a heat flux of $70kW/m^2$ during the heating phase of the experiment. Every C50 column subjected to heating on the "compression face" and a 60% axial load failed during either the heating or the cooling phase, therefore data is only available for the "tension heated" columns.



(a) Ambient experiments



(b) Residual experiments on heated columns

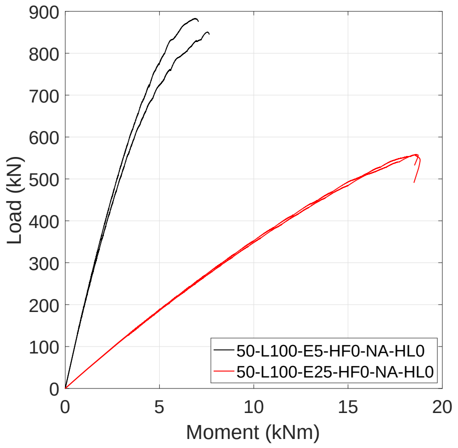
Figure 5.15: Residual load-displacement for C50 concrete columns subjected to a 70kW/m^2 on one face

A similar trend can be observed from the C50, "tension heated" columns to that of the weaker, C30, "tension heated" columns. There is only a slight reduction in the ultimate load and the columns display a peculiar deflection history that would typically not be expected in both cases. The load deflection response of the columns in Figure 5.15b match the behaviour of the weaker C30 columns tested at a high heat flux in Figure 5.11b almost exactly. For a full discussion of this behaviour, see Section 5.4.2.

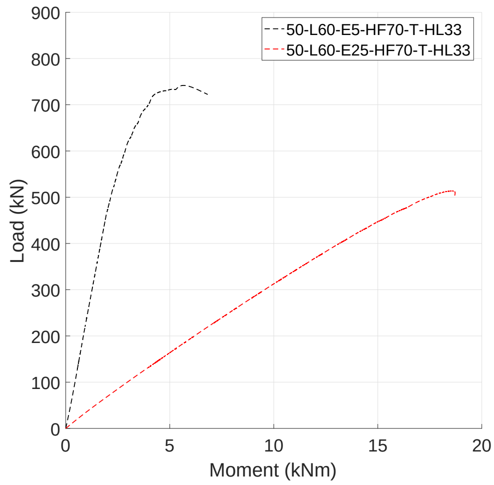
Table 5.7 and Figure 5.16 detail the residual deflections in the columns after cooling back to ambient temperature, and the M-N load paths taken in the residual experiments.

Table 5.7: Residual deflection of C50 concrete columns subjected to a $70kW/m^2$ heat flux

Specimen identifier	Load eccentricity (mm)	Residual deflection (mm)	Effective load eccentricity (mm)
50-L60-E5-HF70-C-HL33	5	Failed	Failed
50-L60-E25-HF70-C-HL33-(1)	25	Failed	Failed
50-L60-E25-HF70-C-HL33-(2)	25 (Repeat)	Failed	Failed
50-L60-E5-HF70-T-HL33	5	-9.8	-4.8
50-L60-E25-HF70-T-HL33	25	2.8	27.8



(a) Ambient experiments



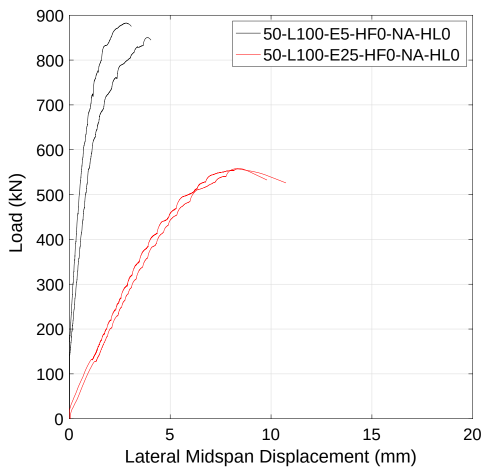
(b) Residual experiments on heated columns

Figure 5.16: Residual M-N load paths for C50 concrete columns subjected to a 70kW/m^2 on one face

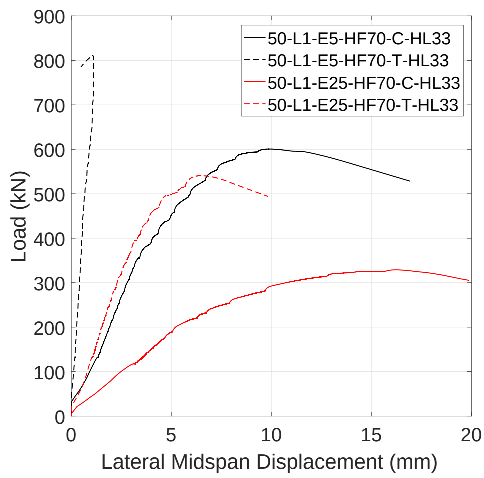
As is detailed in Table 5.7, the residual eccentricity of the columns subject to heating on the "tension face", similar to the C30 concrete columns detailed in Section 5.4.2, appears to be unpredictable. However, as discussed in Section 4.6.5, this is likely a result of the magnitude of the stress on the "compression face" of the column, which is higher in the 5mm load eccentricity scenario. This makes the occurrence of load induced thermal strain more likely. Therefore the heated face in the 25mm eccentricity case expanded as a result of elevated temperature, increasing the effective eccentricity of the load, whereas the heated face in 5mm case contracted as a result of transient thermal creep, reducing the effective eccentricity of the load. This contraction occurred to such an extent that the eccentricity of the load became such that the column essentially transitioned in to being "compression heated". Taking this into account, the behaviour of the columns with Figures 5.15b and 5.16b becomes more explicable. The behaviour of the element subjected to a 5mm eccentricity has been discussed in Section 5.4.2, where the weaker, C30, columns exhibited the same behaviour. The apparent stiffening of the element subjected to a 25mm eccentricity is discussed in Section 5.4.8, where a similar but more pronounced behaviour has been observed for the columns subjected to a larger heated length.

5.4.6 C50 Concrete Columns - 10kN Load

The following figures detail the results of the residual strength experiments conducted on the stronger, C50, reinforced concrete columns exposed to a heat flux of $70kW/m^2$ while subjected a small axial load (10kN).



(a) Ambient experiments



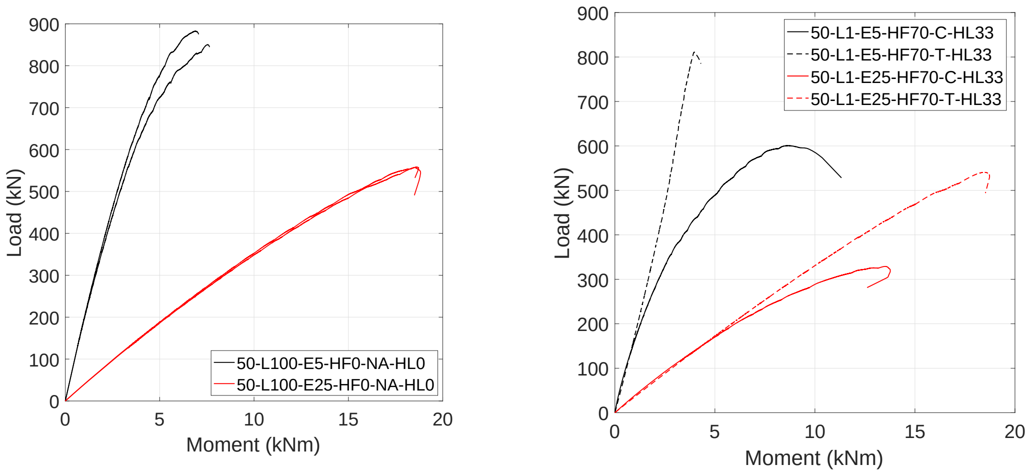
(b) Residual experiments on heated columns

Figure 5.17: Residual load-midspan deflection graphs for ambient temperature strength experiments and of C50 columns subject to a $70kW/m^2$ and a 10kN axial load during heating

Table 5.8 and Figure 5.18 detail the residual deflections in the columns after cooling back to ambient temperature, and the M-N load paths taken in the residual experiments.

Table 5.8: Residual deflection of C50 concrete columns subjected to a $70kW/m^2$ heat flux and a 10kN axial force during heating

Specimen identifier	Load eccentricity (mm)	Residual deflection (mm)	Effective load eccentricity (mm)
50-L1-E5-HF70-C-HL33	5	-0.5	4.5
50-L1-E25-HF70-C-HL33	25	-0.2	24.8
50-L1-E5-HF70-T-HL33	5	1.0	6.0
50-L1-E25-HF70-T-HL33	25	2.7	27.7



(a) Ambient Experiments **(b)** Residual Experiments

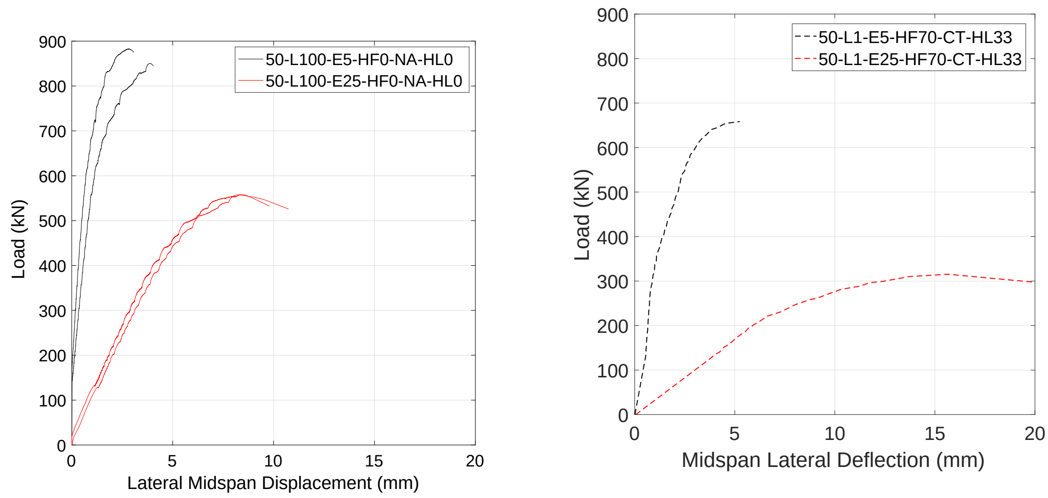
Figure 5.18: M-N load paths for C50 concrete columns subjected to a 70kW/m^2 heat flux and a 10kN axial force during heating

From Figure 5.18 it is clear that the columns loaded with a nominal axial load (10kN) during the heating phase follow the same pattern as for the weaker 30MPa concrete columns, where both the ultimate load and the ultimate moment are decreased at failure. As with the weaker C30 concrete columns, the "tension heated" columns tended to fair better in the residual experiment compared to the "compression heated" column of the same load eccentricity. Considering the 5mm, "tension heated" experiment detailed in Figure 5.18b, it appears to perform remarkably similar to the ambient temperature tests before failing abruptly at a load of 800kN. It should be noted that this column, unlike any of the other columns tested, failed at the supports at the base of the column. However despite unfavourable failure mode, the general trend of the "tension heated" columns behaving in a similar fashion to the ambient temperature experiments holds true.

For a full discussion of the behaviour of the "tension heated" columns with regard to their residual performance, see Section 5.4.3, where this is discussed in relation to the C30 columns which displayed essentially the same behaviour.

5.4.7 C50 Concrete Columns - Double Sided Heating

The following figures detail the results of the residual strength experiments conducted on the stronger, C50, reinforced concrete columns exposed to a heat flux of $70kW/m^2$ on both the "tension face" and the "compression face" during the heating phase of the experiment.



(a) Ambient experiments

(b) Residual experiments on heated columns

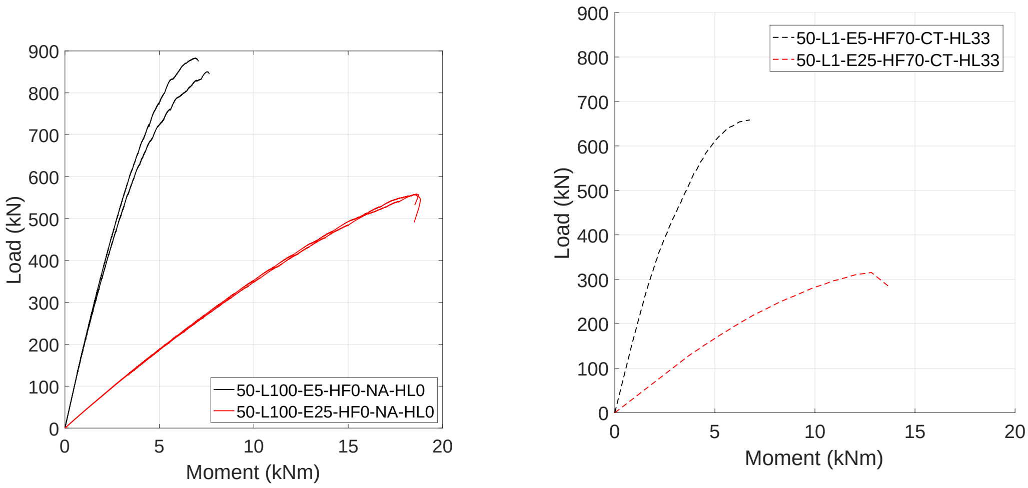
Figure 5.19: Residual load-displacement for C50 concrete columns subjected to a 70kW/m^2 on both the "compression face" and the "tension face"

As a result of the severe damage inflicted on the columns subjected to double sided heating, both columns loaded to 60% of their ambient temperature capacity failed during the heating phase of the experiment. Therefore, only the columns loaded with the nominal 10kN load during the heating phase were tested residually.

Table 5.9 and Figure 5.20 detail the residual deflections in the columns after cooling back to ambient temperature, and the M-N load paths taken in the residual experiments.

Table 5.9: Residual deflection of C50 concrete columns subjected to a $70kW/m^2$ heat flux on both the "tension face" and the "compression face"

Specimen identifier	Load eccentricity (mm)	Residual deflection (mm)	Effective load eccentricity (mm)
50-L60-E5-HF70-CT-HL33	5	Failed	Failed
50-L1-E5-HF70-CT-HL33	5	-0.1	4.9
50-L60-E25-HF70-CT-HL33	25	Failed	Failed
50-L1-E25-HF70-CT-HL33	25	0.2	25.2



(a) Ambient experiments

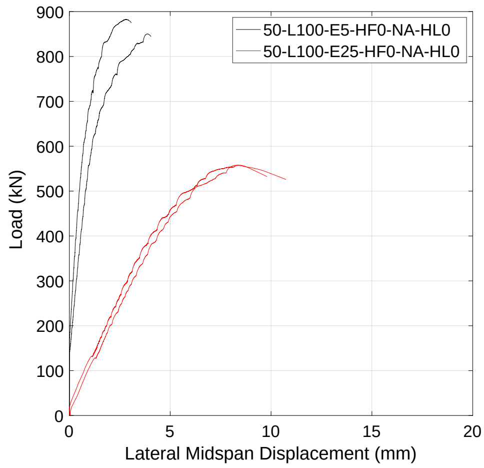
(b) Residual experiments on heated columns

Figure 5.20: Residual M-N load paths for C50 concrete columns subjected to a 70kW/m^2 on both the "compression face" and the "tension face"

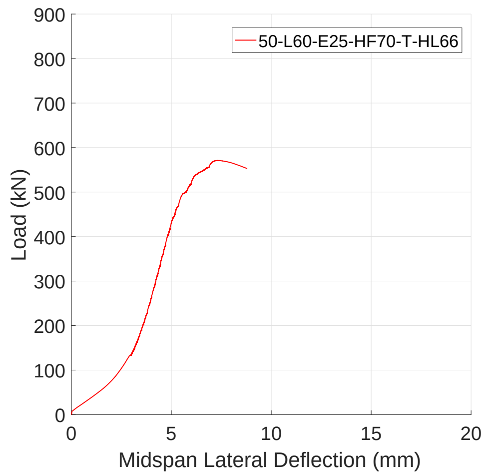
Similar to the columns subject to heating on both the "tension face" and the "compression face" for the weaker C30 concrete columns, the columns detailed in Figure 5.20 see a significant reduction in strength due to the extreme heating regime they were subjected to. As the residual behaviour of the C50 concrete columns is very similar that of the C30 column (low residual deflections etc.), see Section 5.4.4 for a full discussion of this behaviour.

5.4.8 C50 Concrete Columns - Increased Heated Length

Similar to many of the columns subjected to double sided heating, the increased heated length scenario resulted in extreme damage being inflicted on the reinforced concrete columns. As a result, most of the concrete columns subjected to heating over an increased heated length failed during either the heating or cooling stages of the experiment. The only column to survive until the residual phase of the experiment was the "tension heated" column with a 25mm load eccentricity. As discussed in Section 4.10.1, this is most likely a result of the increased volume of concrete undergoing transient thermal creep. In the "compression heated" scenarios this is due to the fact that the heated surface is also the most highly stressed surface. The column subjected to a 5mm load eccentricity while also being "tension heated" does however also subject the heated face to a large compressive stress due to the small eccentricity and almost concentric load. Therefore this column transitioned into being compression heated as a result and failed due to high deflections resulting in large secondary moments during the heating and cooling phase of the experiment.



(a) Ambient experiments



(b) Residual experiments on heated columns

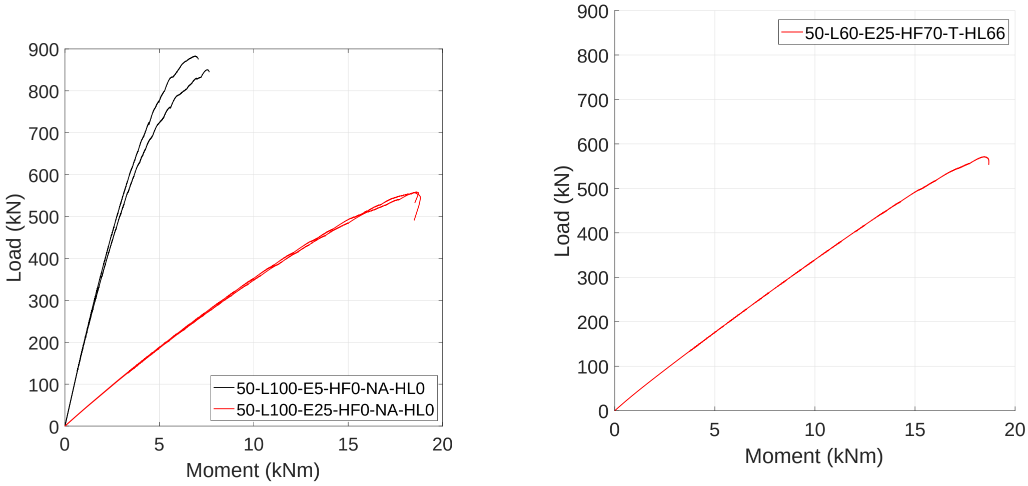
Figure 5.21: Residual load-displacement for C50 concrete columns subjected to a 70kW/m^2 over 2/3 of the length of the heated face

Much like the other "tension heated" columns that were tested residually, the column detailed in Figure 5.21b deflected to a large extent upon first loading and appears to get stiffer as more load is applied. Taking account of the residual deflections present in this column detailed in Figure 5.10, it is clear that the effective eccentricity during the residual experiment is almost the same as that of the eccentricity applied in the heating phase. Therefore the only difference between the ambient temperature experiment detailed in Figure 5.21a and Figure 5.21b is the damage sustained by the heated column on the "tension face". Considering the temperature gradient through the column detailed in Figure 6.11, the column is considerably weaker on the "tension face" than on the "compression face", where the concrete will maintain much of its ambient temperature strength due to the fact that it was not exposed to temperatures as severe as the heated face of the column. Therefore, on first loading, when the entire section of the column is in compression, the strength and stiffness of the heated section is smaller, therefore deflecting to a larger extent. However, as the load is increased, the column bows further resulting in the heated face being subjected to tension, and therefore not contributing to the strength of the section. As the experiment continues and the location of the neutral axis moves farther from the heated surface, the volume of concrete in tension increases and the effective concrete providing the structural resistance to the element is that of the concrete that was not subjected to elevated temperatures. The heated column therefore provides a similar level of resistance to that of the columns tested at ambient temperature because the effective cross section and material properties of the sections are similar.

Table 5.10 and Figure 5.22 detail the residual deflections in the columns after cooling back to ambient temperature, and the M-N load paths taken in the residual experiments.

Table 5.10: Residual deflection of C50 concrete columns subjected to a $70kW/m^2$ heat flux over an extended heated length

Specimen identifier	Load eccentricity (mm)	Residual deflection (mm)	Effective load eccentricity (mm)
50-L60-E5-HF70-C-HL66-(1)	5	Failed	Failed
50-L60-E5-HF70-C-HL66-(2)	5	Failed	Failed
50-L60-E25-HF70-C-HL66	25	Failed	Failed
50-L60-E5-HF70-T-HL66	5	Failed	Failed
50-L60-E25-HF70-T-HL66	25	0.751	25.751



(a) Ambient experiments

(b) Residual experiments on heated columns

Figure 5.22: Residual M-N load paths for C50 concrete columns subjected to a 70kW/m^2 over 2/3 of the length of the heated face

As detailed previously, the effective eccentricity of the column was similar to that of the ambient temperature experiments conducted and the column failed in a very similar manner. Therefore, as detailed in Figure 5.22, the residual experiment on the heated column took a very similar load path in terms of the M-N interaction as the ambient temperature experiments on the reinforced concrete columns. It can be concluded from this that the increased heated length of the columns had little effect on the residual behaviour of the column. This is due to the fact that the columns were heated over an increased length with the same heat flux. Therefore the column experienced elevated temperature over a greater length but the temperature gradient through the depth of the column remained similar. As was discussed, the "tension face" experienced tension and would contribute little to the performance of the column. As a result, the increased temperatures over a greater length are inconsequential as this only occurs on the "tension face" of the column.

5.4.9 M-N Envelope of Damaged Columns

As described throughout Section 5.4, every reinforced concrete section is expected to fail when the combination of axial load and moment (including secondary moments) matched that of the M-N failure envelope for that section. As all of the experiments conducted on elements exposed to the exact same heat flux have a similar temperature gradient and level of damage inflicted, it is possible to plot these experiments against a single M-N failure envelope for the damaged column to compare the predictions with the actual experimental data gathered. For a comparison and discussion of this failure envelope in relation to the sectional model developed, see Chapter 6.

5.5 Discussion of Parameters Investigated

A large volume of data regarding the results of the experiments conducted has been presented in this chapter. Due to the fact that such a large number of parameters were tested, great variation in the behaviour of the columns during the experiments has been observed. However trends were observed in the residual behaviour of the columns which can be physically explained and which, to perform an effective assessment of the structure, must be taken into consideration. A brief summary is therefore provided in relation to the findings of the individual parameters tested:

Concrete Strength: Very little difference was observed between the columns constructed from differing strengths of concrete with respect to their residual performance. The behaviour of the columns tested residually was primarily a function of the damage sustained by exposure to elevated temperatures and the loading conditions (i.e. stress) imposed during heating.

Load Eccentricity: The most important parameter affecting the performance of the columns when changing the eccentricity of the load is the differing stress state on the heated surface. As discussed in Chapter 4 this ultimately determines the residual deflections in the column after cooling because it is one of the two factors that determines the degree of transient thermal creep within the section, changing the load path taken in the residual strength experiment conducted.

Magnitude of Load: Similar to the load eccentricity, the magnitude of the load dictates the stress state in the heated zones during heating, which has the largest effect on the residual displacements experienced. Therefore the magnitude of the load, like the load eccentricity, will determine the load path taken when loaded residually.

Heat Flux: The incident heat flux on a structural element during heating is arguably the most important factor in determining its residual performance. The heat flux imposed will determine both the maximum temperatures achieved in the element and the temperature gradient within. These two parameters will determine both the level of unrecoverable deformation in the element, changing the residual load path taken, and the sectional strength of the column, affecting the maximum axial load and moment that can be resisted after cooling back to ambient temperature.

Side(s) Heated: The side(s) heated, as discussed in Chapter 4, has a significant effect on the residual displacements seen by the elements due to the fact that the "tension face" and the "compression face" have differing levels of stress, therefore affecting the prevalence of transient thermal creep. In addition, the concrete utilised in the residual experiments is that of the concrete closest to the "compression face". Heating the columns on the "tension face" had little effect on the residual performance of the columns due to the fact that this concrete provides little resistance during the later stages of the residual experiments. Whereas heating on the "compression face" resulted in large residual deflections, drastically changing the load path taken when tested to destruction.

Heated Length: It has been concluded in Section 5.4 that increasing the exposed area of the columns has had little effect on the sectional properties of the column and, in some cases, doubling the heated length had no effect on the residual performance. This is due to the fact that imposing an identical heat flux over an increased heated length will result in a similar temperature gradient through the column, which is main factor in determining the sectional properties. Increasing the heated length may however increase the prevalence of phenomena such as transient thermal creep, which will have a large effect on the residual deformation of the element. Therefore, for this specific one-dimensional experimental setup, increasing the heated length will affect the residual displacements of the columns, but leave the sectional properties unchanged.

5.5.1 Implications for Design

As is clear from Section 5.4, the residual performance of the reinforced concrete columns investigated in this experimental test series varies greatly under the three structural arrangements considered (5mm, 15mm and 25mm load eccentricity). This is despite the fact that they are geometrically identical and, in most cases, have been exposed to the same time history of incident heat flux.

After considering the results of the residual strength experiments conducted, it is clear that the current method of determining where the "damaged" concrete is, and adjusting the strength of the cross section appropriately is not adequate to accurately capture the residual response of the elements. The side heated, the magnitude of the load, and the eccentricity of the load all have significant effect on the performance of the column after it has cooled back to ambient temperature.

This is because the concrete does not return to the ambient temperature structural conditions after cooling to ambient temperature. Instead, there are residual (non-recoverable) displacements within the element after it has cooled, resulting in an altogether different structural arrangement than expected i.e. an increased or decreased load eccentricity. This change in the eccentricity of the load, in combination with the weakening of the concrete as a result of being exposed to elevated temperatures, will be the dominating factors in the performance of the column when compared to its ambient temperature response.

It is therefore important to understand the damage inflicted on the element by a "fire" to assess its strength and performance under different conditions. However, to determine exactly how the element will perform, it is also necessary to determine the residual displacements and load eccentricity. It is ultimately these parameters that will determine the load path taken by the column and, as is clear from Section 5.4, the load path taken is critical in determining whether the element will resist a higher axial load/ moment than at ambient temperature, or a considerably smaller axial load/ moment. Incorrectly estimating this, as can be seen from the experiments conducted, can have a drastic, and potentially catastrophic effect of the performance of the reinforced concrete columns.

Chapter 6

Sectional Analysis Model

6.1 Introduction

As part of this research project, an analytical model has been developed to predict the axial-flexural capacity of each of the reinforced concrete columns tested under both heated conditions and residually after heating. This Chapter details the specifics of the models produced and outlines the applications and limitations of the models developed.

Sectional analysis is a well documented and widely applied technique within structural engineering, with detailed guidance widely available on how it should be approached when designing different types of elements [2]. With regard to reinforced concrete columns, as they are normally subjected to a combination of axial load and moment as a result of the load transfer from, for example floor slabs, it is possible to produce axial load-moment (M-N) interaction diagrams for cross sections being designed. Such M-N diagrams provide the designer with an envelope of the possible axial load-moment combinations that the element can support prior to failure. Should a specific combination of load and moment fall outside this envelope, alterations should be made to the section being designed as it is presumed to fail.

Typically, this process is performed for design of cross sections at ambient temperature, where it has been widely validated and proven to be an effective design technique [4]. There is, however, a smaller volume of data available for this method with regard to determining the axial-flexural capacity of a column (or of other structural elements) at elevated temperatures and after exposure to elevated temperatures. This is likely to be partially due to a paucity of experimental data that can be used to validate such models.

Throughout the remainder of this chapter, the classical method of determining the M-N interaction diagram for a reinforced concrete column, and the model developed within this thesis, is described in detail along with the results of both. As the determination of the specific material properties of concrete under different loading and heating conditions is studied extensively in the available literature and is outside the core scope of this project, all material properties are taken directly from Eurocode 2 [4].

6.2 The Classical Section Analysis Model

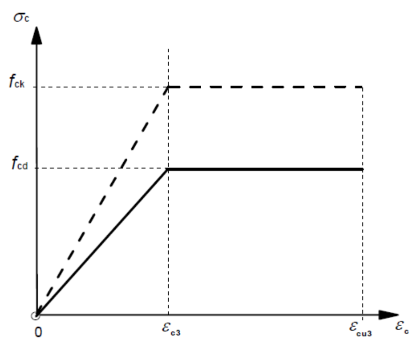
It is possible to determine the axial load-moment (M-N) interaction diagram for reinforced concrete columns at ambient temperature using a straightforward iterative process. This section briefly describes the process involved, and specifically highlights areas where the classical models fail to capture specific behaviours that become important in a calculation of load-moment interactions at elevated temperature.

6.2.1 Assumptions and Calculation Method

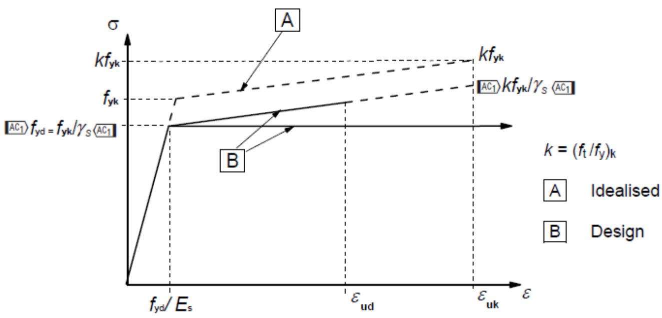
Inherent in the classical M-N interaction model as implemented herein are the following assumptions on the constituent material behaviours:

1. concrete behaves in an ideal brittle manner, as recommended within Eurocode 2 [4]; and
2. steel reinforcement behaves in an ideal elastic-plastic manner as recommended within Eurocode 2 [4].

This idealised behaviour for concrete and steel reinforcement is shown in Figure 6.1. When considering the real material behaviour of concrete, it is considered safe to idealise the material properties as ideal elastic perfectly plastic. The result of this assumption will be conservative based on the fact that the material is assumed to fail before it realistically would. It is also possible because, at ambient temperature with no temperature gradient through the column, as the material properties used for the concrete are the same throughout the entire section. This is an important distinction when moving on to discuss the analysis at elevated temperature.



(a) Concrete material properties



(b) Steel material properties

Figure 6.1: Idealised material properties of concrete and steel reinforcement assumed in the design of concrete sections from BS EN 1992-1-1 [4]

Figure 6.2 details the possible stress and strain distributions through a concrete section as a result of a load-moment combination, given the assumed material properties detailed in Figure 6.1. It can be assumed from this that the reinforced concrete section fails when the extreme fibre of the concrete achieves its ultimate strain (0.0035 from BS EN 1992-1-1 [4]) due to the fact that, using these material properties, this is when the concrete will hold its maximum load. Therefore, to construct a full M-N diagram, it is possible to iterate through every value for the location of the neutral axis, with the extreme fibre strain maintained at 0.0035, determine the force within the section, and take moments about the centre of the section. Each iteration for the location of the neutral axis will provide one point on the interaction diagram. These results can then be collated to produce a full load-moment interaction envelope for any section. The M-N interaction diagram calculated for the reinforced concrete columns cast as part of the experimental test series described in Chapter 3 have been provided in Figure 6.3 using the author's model that was developed for this thesis. The model developed to demonstrate the classical model was created using MATLAB.

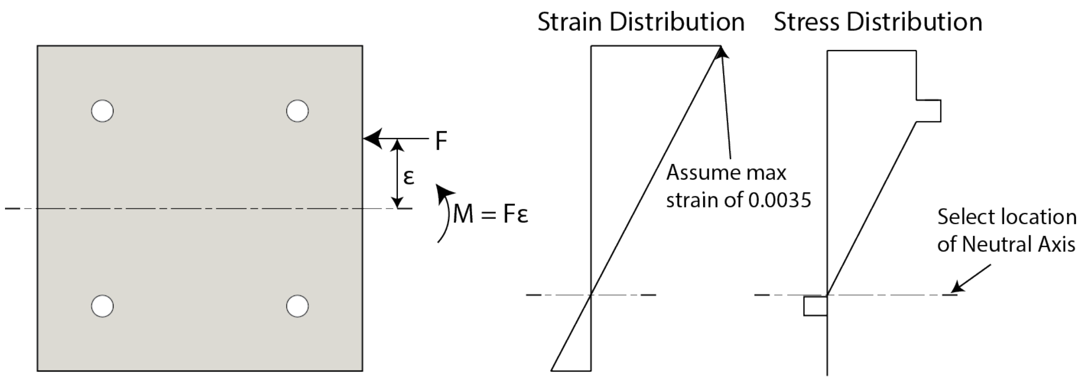


Figure 6.2: Example stress-strain distribution within a section given the idealised material properties assumed within BS EN 1992-1-1 [4]

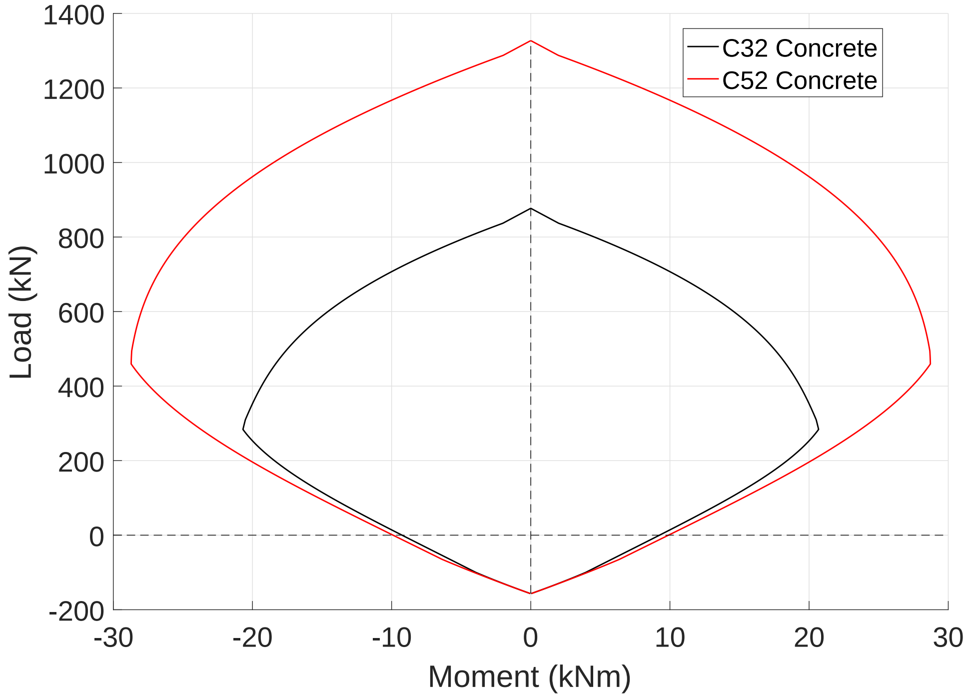


Figure 6.3: M-N interaction diagram for C30 and C50 reinforced concrete columns cast as part of this research project (see Chapter 3)

Due to the common and conservative assumption that concrete bares no resistance in tension, it is clear from Figure 6.3 that the columns constructed from both types of concrete achieve the same level of resistance under pure tension due to the fact that the same volume of steel reinforcement was used in the design of the columns cast. In addition, the resistance of the columns under both negative and positive moments is identical (for each column type) as a result of the symmetrical design and homogeneous material properties assumed.

6.2.2 Limitations at Elevated Temperature

The model outlined above provides engineers with a method of relatively easily estimating the M-N interaction diagram for any given section at ambient temperature. Given the inherent assumptions in the method, it will also generally provide the engineer with conservative results for the axial-flexural capacity of any section. However, the task of determining the M-N interaction diagram becomes significantly more challenging when the temperature within the section increases above ambient temperature, especially if this increase in temperature is non-uniform through the section i.e. if there is a temperature gradient through the section. This is the result of the fundamental assumption used in ambient temperature M-N interaction diagram development that the columns maximum capacity is defined as the point when the extreme fibre of the concrete reaches its ultimate strain; this may not be the case when the concrete has been exposed to elevated temperatures due to the softening of concrete at elevated temperature, and that fact that the extreme fibre may also be the hottest part of the concrete element (for columns in particular).

Unlike BS EN 1992 Part 1-1 [4] which primarily deals with the design of concrete structures at ambient temperature, BS EN 1992 Part 2-2 [2] recommends a slightly different approach to calculating the stress-strain relationship of concrete to allow for design and analysis at elevated temperatures. It is recommended that a descending branch of the stress-strain curve be introduced, which negates the previous assumption that the column reaches its maximum capacity upon the extreme fibre reaching a compressive strain of 0.0035. This is a result of the fact that the concrete beyond the ultimate strain may still contribute to the capacity of the column, meaning that ultimate load carrying capacity of the concrete column could occur with the 0.0035 strain condition being located deeper within the cross-section, as opposed to being dictated by the strain at the extreme fibre. This results in the need for an iterative analysis procedure.

As regards to concrete, as the temperature of the concrete increases, the ultimate strength of the concrete decreases. When applying this to a reinforced concrete section with a temperature gradient, the result is a non-linear stress distribution throughout the section (see Figure 6.4) unlike the piecewise linear stress distribution detailed in Figure 6.2. Since the stress-strain properties of the concrete are no longer linear, it can no longer be assumed that the column fails when the ultimate compressive fibre reaches a presumed ultimate strain of 0.0035. Given the hotter, weaker concrete at the surface of the section, the concrete will start to soften at the surface, however the concrete deeper within the section will be capable of resisting higher stresses due to the fact that it remains at a lower temperature. As a result, it is no longer possible to simply iterate through all of the possible locations of the neutral axis to find the M-N interaction diagram.

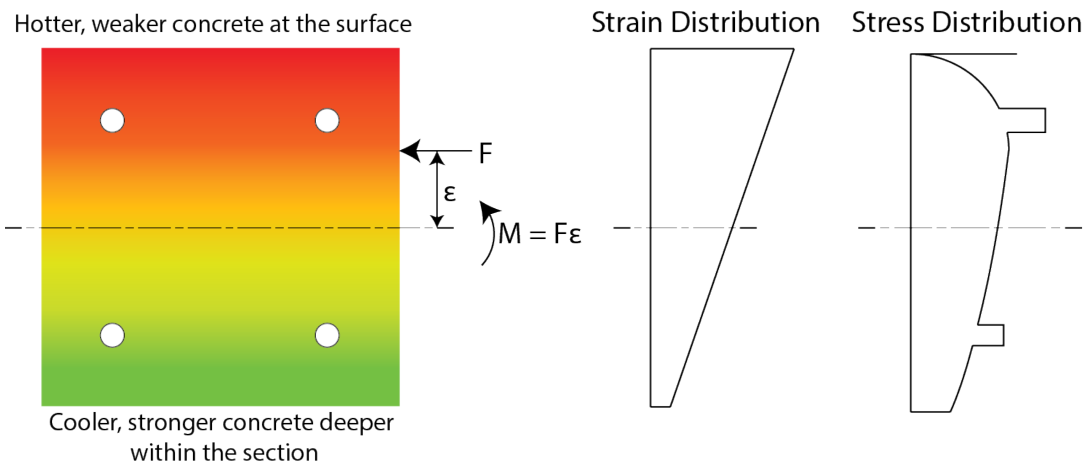


Figure 6.4: Possible stress distribution within concrete section due to exposure to elevated temperatures

To summarise, the following additional items must be taken into consideration when calculating the M-N interaction diagrams for reinforced concrete columns at elevated temperatures, as compared with the classical method used at ambient temperature.

1. The mechanical properties of concrete and steel change at elevated temperatures and can no longer be idealized in the manner used at ambient temperature.
2. As a result of the changing mechanical properties, the column cannot necessarily be assumed to have reached its peak axial-flexural load carrying capacity when the extreme fibre reaches the ultimate strain.

6.2.3 500^o Isotherm Method

To extend the applicability of the classic model to calculating the resistance of a section exposed to elevated temperatures, the 500^oC Isotherm Method is recommended in Annex B of BS EN 1992-1-2 [2]. This method quite simply involves implementing the classic model detailed in Section 6.2 with a reduced cross sectional area of concrete. It is assumed that all concrete above a temperature of 500^oC does not contribute to the load bearing capacity of the section. Whereas concrete below a temperature of 500^oC retains it's ambient properties of strength and elasticity [2]. Therefore the material properties of concrete at elevated temperature detailed in Figure 2.5 are not required to be taken into account. The reduction in strength of the section comes from the reduced cross sectional area of the concrete. One item that is however not taken into account when implementing this method is the increased ductility of the concrete between 100^oC and 500^oC before it is assumed to have failed. A comparison of the 500^oC Isotherm Method from BS EN 1992-1-2 [2] and the method developed herein by the author has been detailed in Section 6.6, to determine what effect, if any, this has on the axial-flexural capacity predicted for the sections.

6.3 The Section Analysis Model Developed

In Section 6.2, the classic sectional analysis model used in the calculation of M-N interaction diagrams for reinforced concrete columns was summarised. It was concluded that this method of calculation is not valid when analysing concrete sections exposed to elevated temperatures. Therefore a sectional analysis model has been created by the author to take account of the items that are not considered in the classical approach. The model produced is an analytical, sectional analysis model written in the programming language C++. It is based upon the classical model described in Section 6.2 with some notable differences and extensions:

1. The material models for concrete and steel reinforcement at elevated temperature described in BS EN 1992-1-2 [2] have been used as opposed to the ambient temperature properties recommended in BS EN 1992-1-1 [4].
2. The section is discretised into an elemental mesh to account for the differing material properties through the section due to the presence of a thermal gradient.

The process implemented is similar to that of the process competed by El-Fitiany and Youssef in 2014 [63]. The process of calculating the maximum axial load/ bending moment for every combination of load is the same as with the classical model with the exception that no failure is assumed in the section based on limiting compressive strain. Unlike the classical model, the strain distribution within the section is not known at the point of peak axial-flexural loading, therefore a second set of iterations must be undertaken changing the strain distribution within the section. This allows the "load path" to be plotted for each location of the neutral axis as opposed to simply assuming failure at a certain strain. When the load paths for every iteration are plotted together, the maximum M-N combinations can be plotted by taking the envelope of all of the iterations. This multiple iterative process has been detailed in Figure 6.5 in the form of a flow chart.

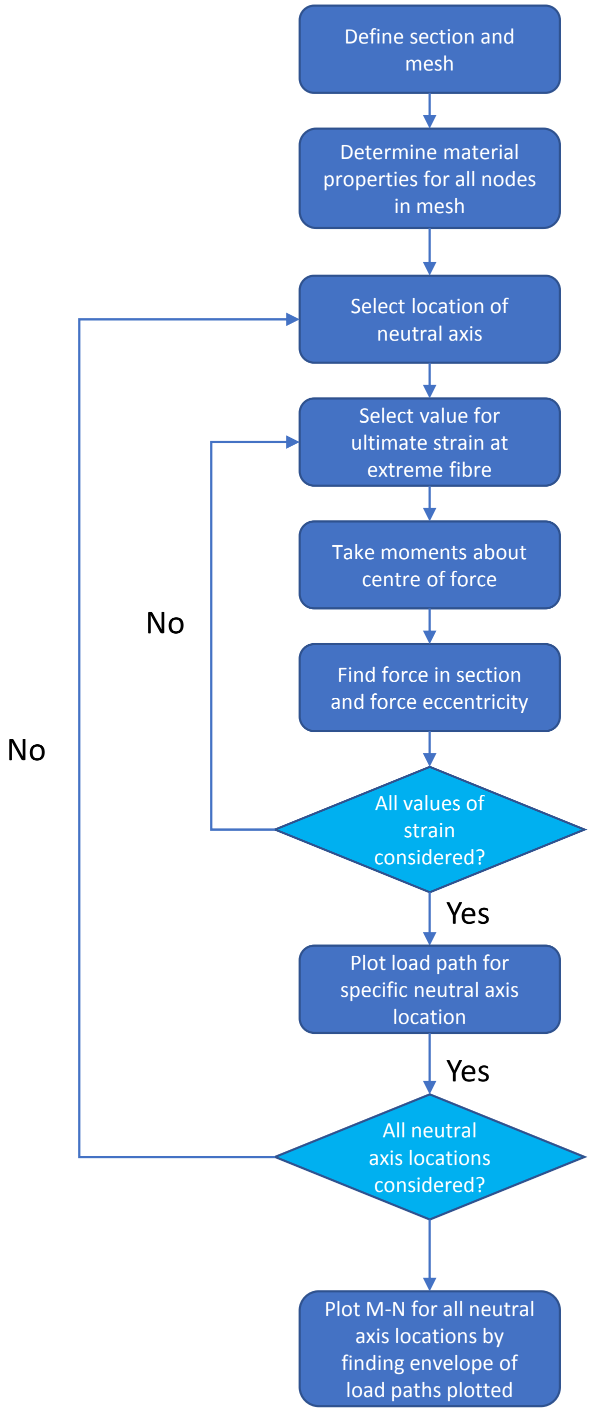


Figure 6.5: Process flow chart showing how M-N interaction diagrams are developed for reinforced concrete sections at elevated temperature

It should however be noted that this process is computationally expensive and would require software developed specifically for this task. The following sections detail an example calculation of a reinforced concrete column exposed to elevated temperatures. The process described in Figure 6.5 has been followed and all assumptions and limitations have been discussed throughout.

6.4 Comparison of Classical Model and the Elevated Temperature Model when Applied to Ambient Temperature Analysis

Before presenting the results of the analysis carried out on the reinforced concrete columns tested in Chapter 4, the two models described in the previous sections have been compared at ambient temperature for the columns used as part of the experimental test series described in this thesis. The results of this comparison are detailed in Figure 6.6.

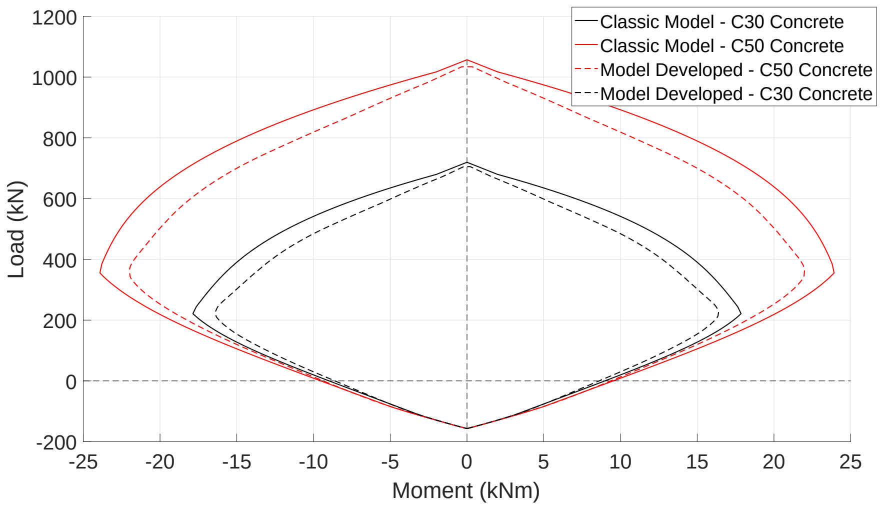


Figure 6.6: Comparison of the sectional analysis model developed with the classic sectional analysis model for the C30 and C50 concrete used as part of the experimental test series

Figure 6.6 shows, when subjected to tensile forces, the two models predict similar load-moment interactions. This is again due to the fact that both models use the same material properties for steel at ambient temperature and both assume that concrete provides no strength in tension. Therefore, when in tension, the models, despite the slightly different assumptions of when failure occurs, the values returned are the same because the concrete in the section is mostly ignored when in tension, and the values are computed from the contribution of the steel only.

However, this changes as the compressive force increases. The elevated temperature model developed as part of this thesis underestimates the moment contribution at peak load compared to the classical model. This can be explained however when looking at the assumptions for the material properties of the concrete for both models. As discussed in Section 6.2.1 and detailed in Figure 6.1,the classical model uses simplified bi-linear material properties recommended for ambient design in BS EN 1992-1-1 [4], which assume that concrete behaves in a linear elastic, perfectly plastic manner until failure at a strain of 0.0035. Therefore, when calculating the force contribution of the concrete in the section, much of the concrete will be strained to a point where it provides a resistance of f_{ck}. Whereas the model developed uses material properties for design of concrete at elevated temperature as discussed in Section 6.2.2 in which the concrete reaches a stress of f_{ck} just before "failure".

This difference in the material properties between the two models will slightly different strain profiles assumed at failure. This is turn results in the slightly lower estimated values for the M-N interaction under compressive loads.

6.4.1 Applicability and Limitations

It is important to note that there are a number of limitations to this process as described. In calculating the M-N interaction diagrams for each section, only the maximum combination of load and moment that the column can withstand are calculated. The real behaviour of the column as a result of different phenomena such as transient thermal creep are not explicitly taken into consideration (although they may be implicitly included in the BS EN 1992-1-2 material properties for concrete at elevated temperature [4]). These phenomena will not effect the calculation of an M-N interaction diagram of the section, but they can be expected to effect how rapidly the element approaches the failure envelope. For example, in the experiments conducted as part of this thesis and described in Chapter 4, which this model is being compared against, transient thermal creep was indirectly observed. As a result of this material behaviour, the deflections in the columns subjected to heating on the compression face increased. This resulted in the columns exhibiting larger axial and lateral deflections, despite being subjected to identical loads and load eccentricities for the duration of the experiment. As a result of the increased deflections, the secondary moment induced in the sections would increase. None of this system behaviour is captured in the production of an M-N interaction diagram, as discussed later in this chapter.

6.4.2 Example Calculation

Given the process outlined in Section 6.3, a full calculation of the load-moment interaction diagram for the following section subject to a temperature gradient has been provided to demonstrate the output of the model developed.

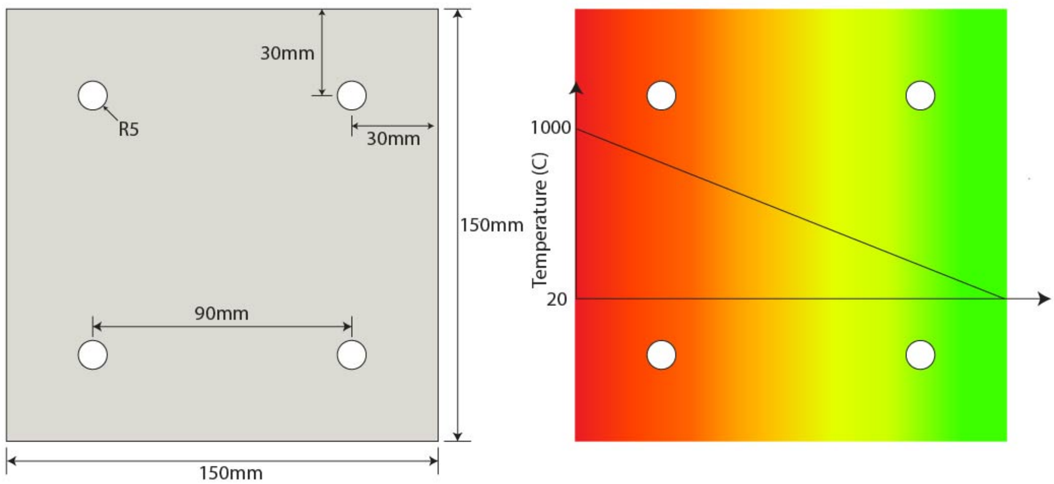


Figure 6.7: Reinforced concrete section and temperature gradient analysed

It should be noted that the temperature gradient through the section has been linearised for simplicity. This example calculation is not an analysis of any of the experiments described in Chapter 3.

Section Discretisation and Input Parameters

Given the section detailed in Figure 6.8,the concrete column is discretised into a mesh of vertical slices of 1mm thickness. The temperature of every 'node' in the analysis is defined as the average of the temperature over the slice area. The materials properties for concrete and steel at elevated temperature are taken directly from BS EN 1992-1-2 [2], as discussed in Section 6.3.

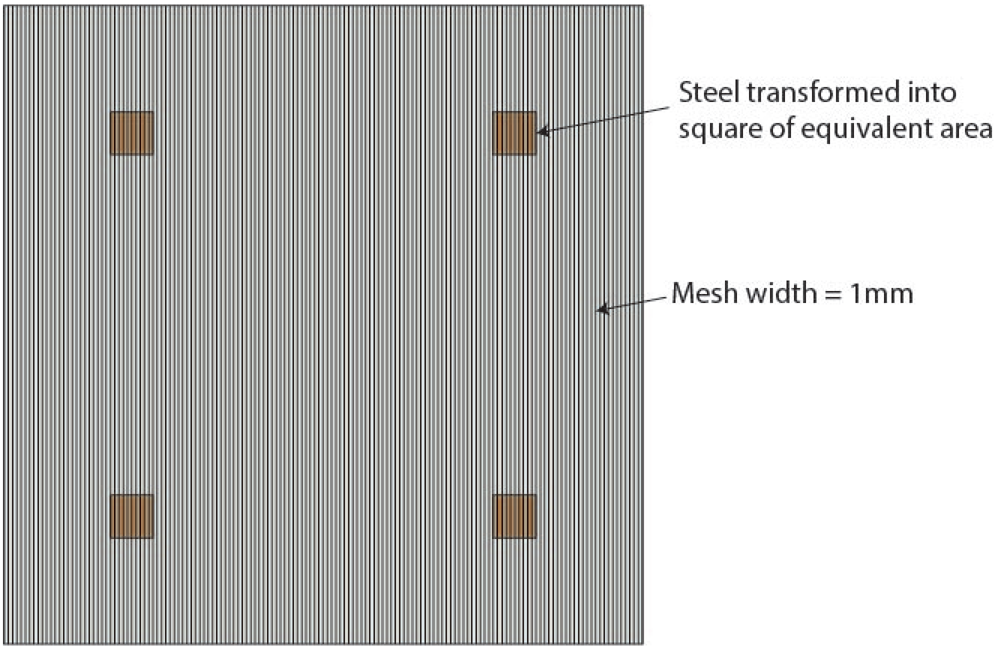
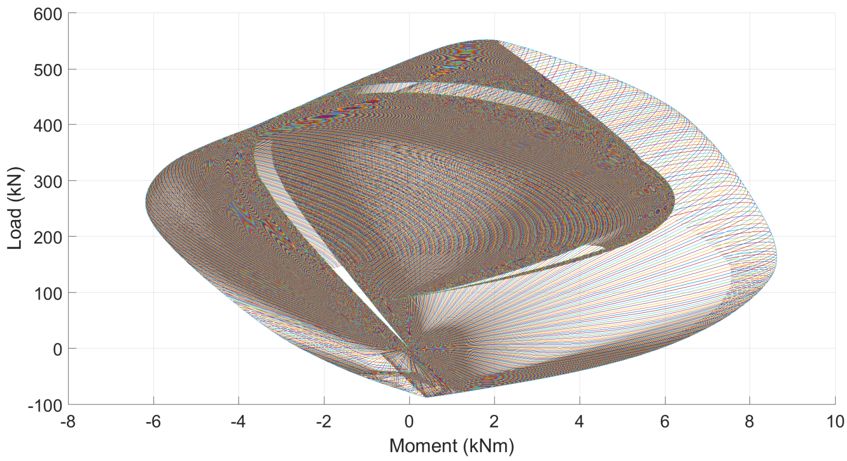


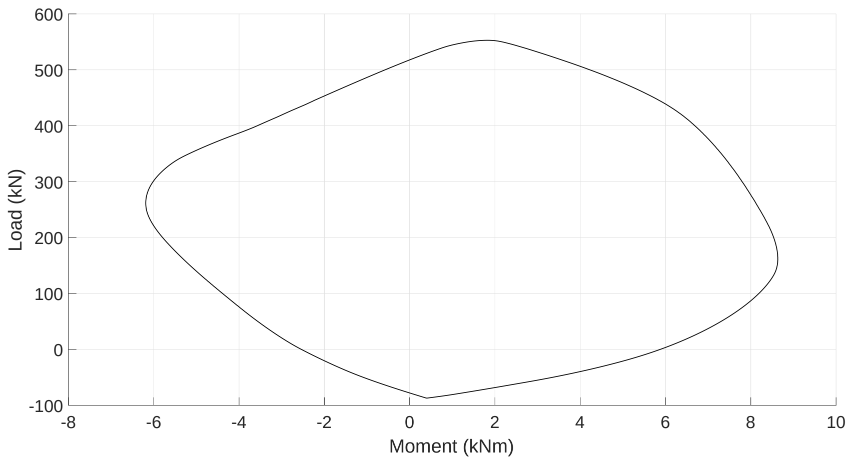
Figure 6.8: Mesh Details

Constructing the Interaction Diagram

To construct the interaction diagram, it is necessary to plot the load paths for each of the iterations for the location of the neutral axis. As it is difficult to predict the point at which the column fails using this method, a brute force approach is required to calculate the interaction of axial force and moment under every possible combination, and then plot the envelope. This process has been detailed visually in Figure 6.9.



(a) Every load path for each neutral axis location in calculating the axial-flexural interactions for the example column at the assumed elevated temperature condition shown in Figure 6.7



(b) M-N interaction envelope (diagram) for all load paths calculated for the example column at the assumed elevated temperature condition

Figure 6.9: Construction of the M-N interaction envelope from the individual load paths calculated

Figure 6.9a details the results of the model detailing every load path for all of the neutral axis locations considered. Figure 6.9b details the envelope of Figure 6.9a detailing the maximum M-N interactions for the reinforced concrete column analysed. It can be seen from Figure 6.9a that many of the possible load paths plotted span the entire width of the plot. This is the result of the fact that it is assumed that the column does not fail, yet continues to bend to the limit imposed by the user. This however is inconsequential due to the fact that, after reaching the limit of the M-N envelope, the remaining calculations all fall within the limits of the other load paths plotted because that particular neutral axis location has achieved it's maximum.

It is also clear from Figure 6.9 that the new interaction diagram for the column after being exposed to the temperature gradient detailed in Figure 6.7 is skewed compared to the envelope plotted in Figure 6.6. In addition to this, the maximum load and moment achieved have both been reduced as a result of exposure to elevated temperatures. As a result of the temperature gradient through the column, the material properties of the concrete and steel differ through the depth of the section. This results in the skewing of the M-N interaction envelope as is seen in Figure 6.9.

Unlike the interaction of the columns at ambient temperature, the maximum tension force that can be carried by the column occurs in conjunction with a positive moment, however inducing a negative moment reduces the maximum resistance of the column to tension forces. As with the classic model, it is assumed that concrete provides no resistance in tension. Therefore, in the tension condition the concrete provides not benefit to the column. All of the stress in the section is distributed between the four reinforcing bars. As a result of the temperature gradient in the column, the two reinforcing bars closer to the heated surface are weaker that those further from the heated surface. Therefore, under pure tension, the maximum force held by the reinforced concrete column is the sum of the resistance of the four reinforcing bars. If a positive moment is however induced ("compression heated"), the weaker reinforcing bars are being compressed slightly, allowing the stronger reinforcing bars take a larger share of the tensile force. As a result, inducing a positive moment increases the total resistance of the section. The opposite is however true when a negative moment is induced in the column, the stronger reinforcing bars are being slightly compressed and the tension in the weaker bars is increased. As the weaker bars have already achieved their maximum level of stress, the column will fail and the total resistance will be reduced compared to pure tension or the positive moment case.

Under compression, it clear from Figure 6.9b that a similar interaction occurs between the moment and the force. The maximum compression force that can be held by the column also occurs in combination with a positive moment and, inducing a negative moment in the section reduces the resistance of the column. Under pure compression, the maximum force carried by the column will occur at a strain when the stronger concrete has started to fail to utilise the maximum volume of concrete. Therefore, inducing a positive moment allows the section to utilise the stronger concrete and the weaker concrete at the same time due to the fact that the weaker concrete achieves its maximum stress at a higher strain. Therefore, creating a strain profile where the weaker concrete is strained higher than the stronger concrete will utilise the strength of the maximum volume of concrete. In comparison, a negative moment will result in higher strains in the stronger concrete, resulting in the column reaching its peak axial-flexural capacity before utilising the weaker concrete in the section.

6.5 Comparison with Experiments

Section 6.3 and Section 6.4 have provided an overview of the high temperature M-N interaction model developed by the author and a comparison with the classical model used at ambient temperature in calculating the M-N interaction for reinforced concrete sections. This section will now consider the experiments undertaken in Chapters 4 and 5, and compare the data collected to the predictions made by the model described (as well as with the 500C Isotherm Method of BS EN 1992-1-2 [2]).

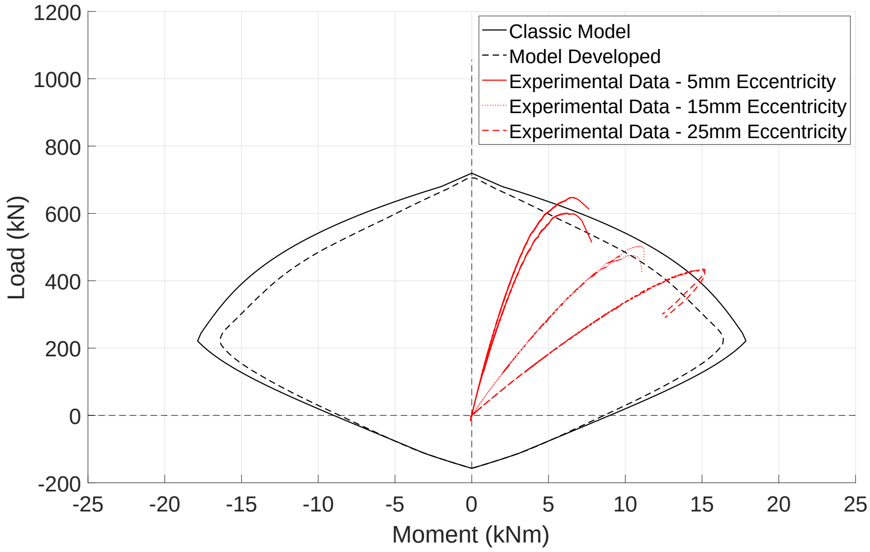
Assumption and Model Parameters Implemented

As discussed in Section 6.3, all material properties used in the calculation have been taken directly from BS EN 1992-1-2 for the specific class of concrete used in the experiments. As discussed in Section 5.2.4, the real temperature gradient through the columns will be two dimensional, as illustrated in Figure 5.3. However the model developed at the time of writing does not accept two dimensional temperature gradients and all calculations are completed using the assumption that the temperature gradient through the column is one dimensional only. Therefore, for each of the reinforced concrete columns detailed in the following analysis, the temperature gradient through the column has been taken using the thermocouples cast into the columns at mid-height, on the centre line (where failure occurred for most columns). As the temperatures at the sides of the columns will be lower than what has been assumed in the columns, all calculations conducted should, in theory, be conservative due to the fact that a larger volume of weaker concrete has been assumed. As the temperatures were only measured on the centreline of the column during the experiments, it is not possible at this point to determine to what degree this may effect the calculation. As the model can be easily adapted to make it 2D as opposed to 1D, an additional piece of work may that may be conducted is to determine to what extend the assumption of uniform temperatures on each slice has effected the calculation.

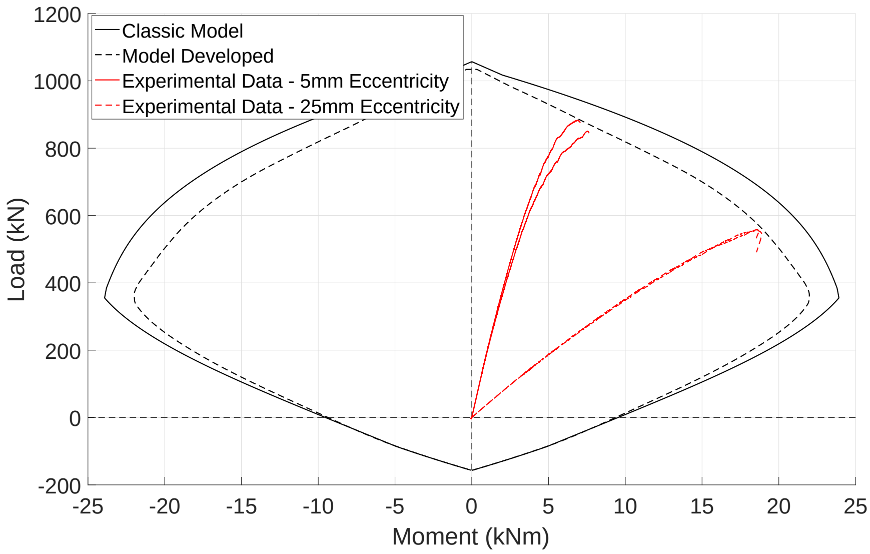
It is recommended in BS EN 1992-1-2 that any increase in strength in the concrete as a result of cooling should be ignored in the calculation of the resistance of concrete elements in fire therefore no increase in strength has been assumed as the concrete cools. In addition to this BS EN 1994-1-2 recommends an additional 10% reduction in the strength of concrete after cooling back to ambient temperature conditions. There is however no guidance as to when this 10% reduction should be applied when the concrete is in the cooling phase. To the authors knowledge there is no definitive research in this area. Therefore, all of the reinforced concrete columns tested in this experimental test series have been compared to the model produced using the high temperature material properties of concrete, even after cooling back to ambient temperature (i.e. no additional strength reduction in the concrete has been assumed after cooling). This determines the ability of the model to predict the resistance of reinforced concrete before, during and after a fire using the recommended material properties of BS EN 1992-1-2 [2] (where there is no recommendation for additional strength reduction).

6.5.1 Concrete Columns at Ambient Temperature

As described in Chapter 4, each of the concrete columns cast as part of this experimental test series were tested to destruction under each of the load conditions investigated to determine the ambient resistance of the columns. As the classical model is applicable to columns tested under ambient conditions, the results of these experiments have been plotted alongside the predictions of both the classical model and the high temperature model developed when applied at ambient temperature conditions. This comparison is shown in Figure 6.10.



(a) C30 concrete columns



(b) C50 concrete columns

Figure 6.10: Comparison of the experimental results (midspan axial-flexural load paths) with the predictions of the sectional analysis M-N interaction models at ambient temperature

In predicting the strength of the reinforced concrete columns in Figure 6.10, both the classical model and the high temperature model developed provide reasonable predictions of axial-flexural capacities when compared against the experimental results. It is, however, clear that the high temperature model developed as part of this thesis tends to slightly under predict the strength of the columns for the weaker C30 concrete columns cast (by 8% - 15%), whereas the failure envelope seems to be very accurately predicted for the stronger C50 concrete columns cast (within 1%). There may be a number of reasons for the more accurate prediction of the failure envelope for the stronger reinforced concrete columns. A relatively small number of concrete cylinders (five) were tested for each of the concrete mixes to determine their 28-day compressive cylinder strength. As is expected when testing concrete, a range of concrete strengths was measured from these experiments, and an average taken. The samples taken for the stronger columns may have provided a more representative average concrete strength than the weaker cylinders. Alternatively, the model itself may always under predict the strength of the sections. The stronger concrete columns could be more prone to failing as a result of buckling of the reinforcement due to the higher loads present during testing, therefore appearing to match the predictions more reliably despite having a slightly different failure mechanism or failure-instigating mechanism which is not explicitly accounted for in the model. There are a number of reasons for this however, based upon the sample the ten experimental results presented in Figure 6.10, it appears that the model predicts the failure envelope of the reinforced concrete columns at ambient temperature reasonably. This is more or less to be expected given the volume of prior work completed in this area by others, its widespread acceptance within the structural engineering community, and the guidance provided throughout BS EN 1992-1-1 [4] and BS EN 1992-1-2 [4].

6.5.2 Columns at Elevated Temperature

In comparing the results of the elevated temperature experiments conducted with the predictions of the elevated temperature model produced as part of this thesis, it is first necessary to consider the heating phase in which the column failed. As the experiments conducted took place in three distinct stages; heating, cooling, and residual performance. As discussed in Chapter 2, the behaviour of concrete after exposure to elevated temperature differs depending on which of these three stages it is in. It is therefore not possible to use a "one size fits all" solution in predicting the resistance of the columns after a fire. There is however a paucity of work allowing for tailored approach to the calculation of the resistance of the sections depending on the conditions after a fire. All of the columns tested have therefore been compared against the predictions of the models assuming that the material properties of the concrete are that of the highest temperatures achieved at a given location prior to failure regardless of whether the column had cooled back to ambient temperature prior to being loaded to failure.

As discussed in Section 4.3, the temperature gradients through all of the reinforced concrete columns during the experiments were similar due to the fact that each of the columns were exposed to essentially identical heat fluxes for the same durations. There were some differences observed in the temperature gradients through the columns that deflected in opposing directions (towards or away from the heat source). However, as discussed and detailed in Figure 4.4, the differences in temperatures occurred primarily at the surface of the columns, where the concrete was well beyond the temperature at which it provides any significant benefit from a structural perspective. The temperatures of the concrete deeper within the columns, where most of the structural resistance is developed, is more consistent between the experiments. Therefore, as the columns can be assumed to have essentially the same internal temperature gradients, every experiment conducted under the same conditions should be suitably represented by a single interaction envelope. Where the temperature gradient through the columns was significantly different (greater than 50^{o}C), an additional failure envelope has been calculated to ensure a like-for-like comparison of the experiments and the models being compared.

The temperature gradients used in the calculation of the failure envelope of the columns at each of the heat fluxes investigated has been taken from the thermocouples cast into the concrete columns themselves. The temperature between two adjacent thermocouples has linearly interpolated to provide an estimate of the gradient through the column. An example temperature gradient used in the calculation of the failure envelope of the columns has been detailed in Figure 6.11.

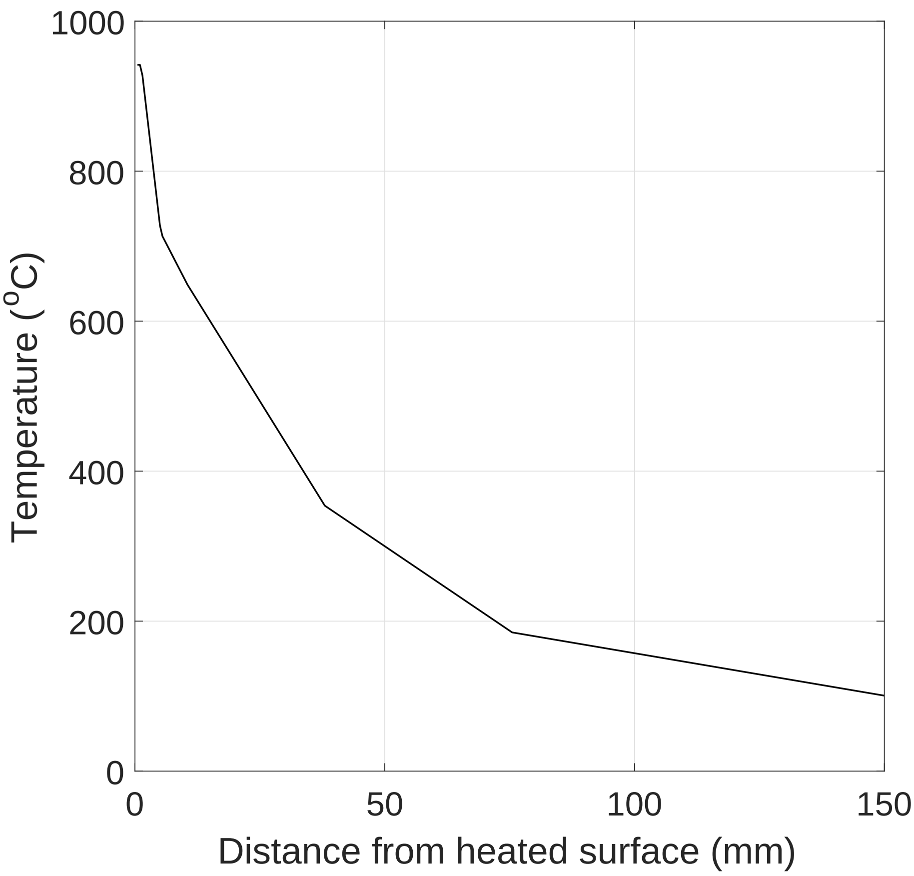


Figure 6.11: Maximum temperature gradient used in calculating the M-N interaction diagrams of columns exposed to a 70kW/m^2 heat flux on one side only

Using the temperature gradient detailed in Figure 6.11 and the material properties recommended in BS EN 1992-1-2 [2], the failure envelopes shown in the following figures were predicted for the reinforced concrete columns tested as part of this experimental test series.

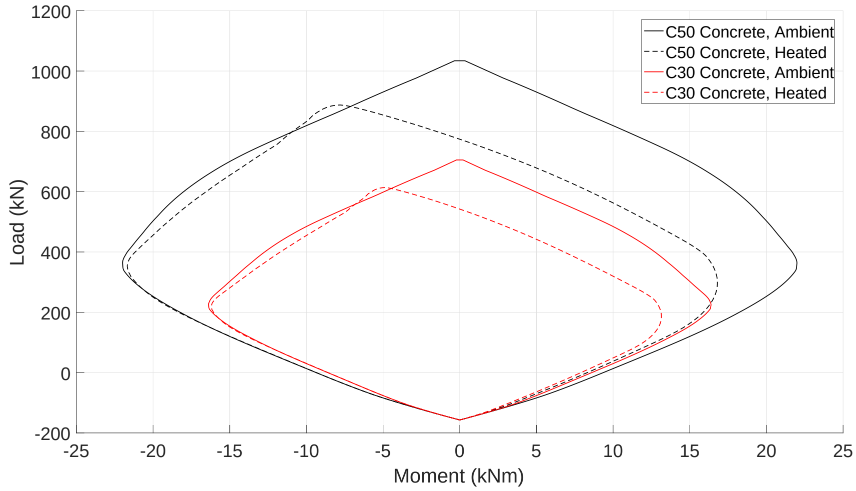


Figure 6.12: Comparison of the predicted M-N envelope for columns at ambient temperature and after being heated to the temperature gradient specified

Similar to the M-N gradient detailed in the example calculation in Section 6.4.2, it can be seen that the M-N envelopes of the columns tested are both weaker than the ambient envelope and skewed to one side. This is a result of the non-symmetrical damaged caused by heating the columns on one side only, the mechanics of which have been discussed in more detail in Section 6.4.2.

6.5.3 Columns Failing During Heating

Figure 6.13 and Figure 6.14 show a comparison of the elevated temperature M-N interaction model developed and the actual experimental data (axial-flexural load path) collected for the reinforced concrete columns that failed during the heating phase of the experiments. Since each of the columns that failed during heating failed at a different time after the radiant panels were switched on, each has to be analysed individually due to the fact that the temperature gradient through the columns was slightly different at failure in each case.

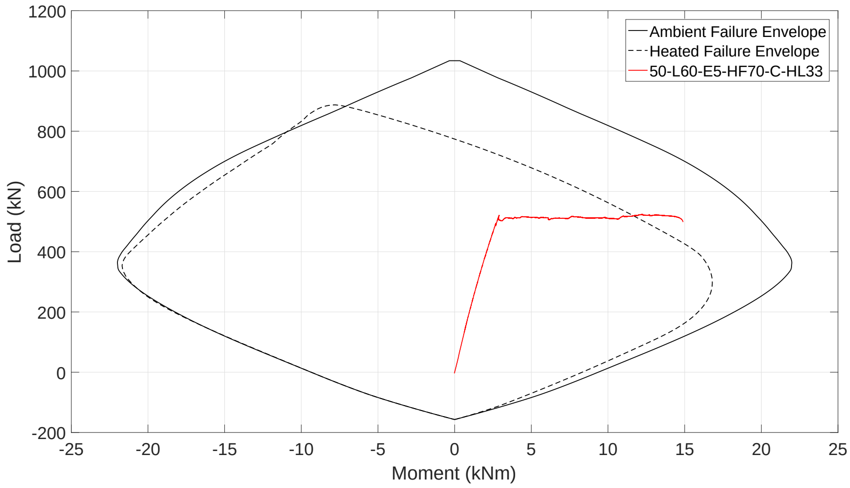


Figure 6.13: Comparison of the experimental results with the predictions of the sectional analysis model for column 50-L60-E5-HF70-C-HL33

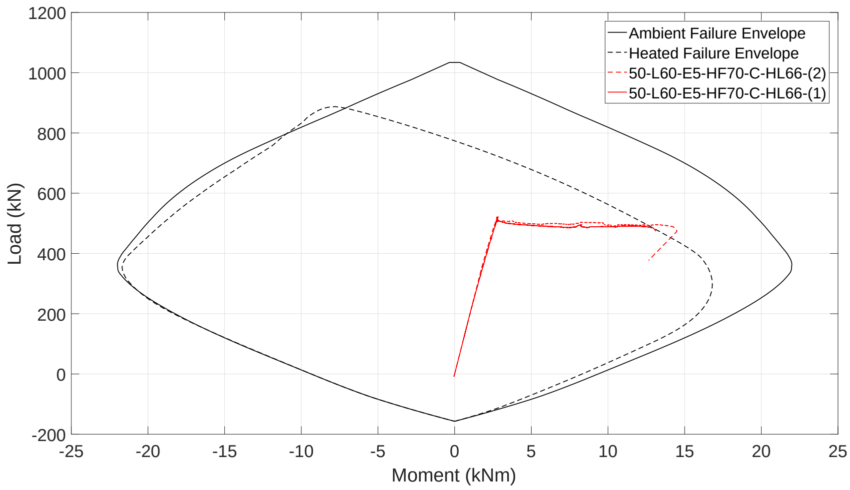


Figure 6.14: Comparison of the experimental results with the predictions of the elevated sectional analysis model developed for columns 50-L60-E5-HF70-C-66(1) and 50-L60-E5-HF70-C-66(2)

For the reinforced concrete columns that failed during the heating phase of the experiment, it is clear that the model used predicts the point of failure for the three columns fairly accurately, slightly underestimating the point of failure for experiment 50-L60-E5-HF70-C-HL33 and by 8%. The prediction for the repeat tests 50-L60-E5-HF70-C-HL66 is within 0.5% for one and 5% for the other. It should however be note that it is not possible from here to assume that the model is capable of predicting the point of failure of any concrete section due to the fact that there are only two test comparisons available from this experimental test series. It does however provide a positive basis for predictions to be used to predict and compare against other experimental test series conducted in the future.

6.5.4 Columns Failing During Cooling

Figures 6.15, 6.16 and 6.17 give comparisons of the elevated temperature model developed and the actual experimental data collected for the reinforced concrete columns that failed during the cooling phase of the experiments. Note that despite failing in the cooling phase of the experiments, any increase in strength of the concrete as a result of cooling has been ignored in the model, in accordance with the design guidance [2]. It should be noted that the material properties of the steel reinforcement are assumed to take the material properties of steel at a given temperature as detailed in BS EN 1992-1-1 [4] i.e. the steel material properties recover upon cooling.

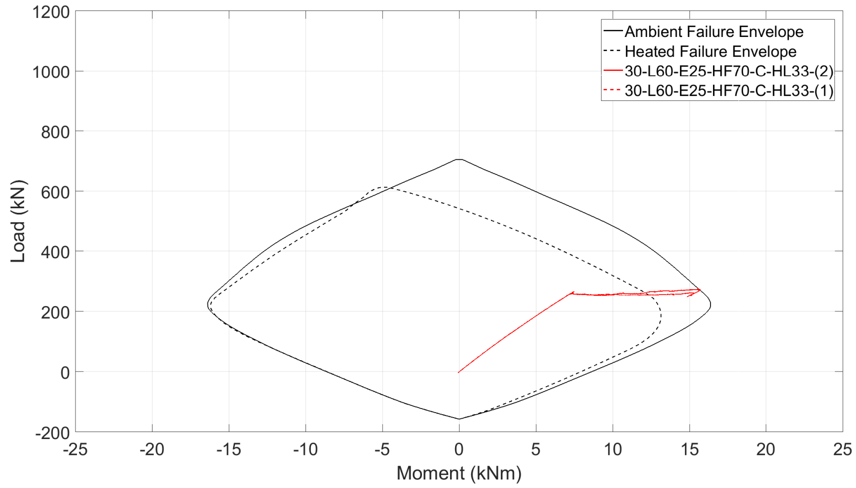


Figure 6.15: Comparison of the experimental results with the predictions of the sectional analysis model for columns 30-L60-E25-HF70-C-33-(1) and 30-L60-E25-HF70-C-33-(2)

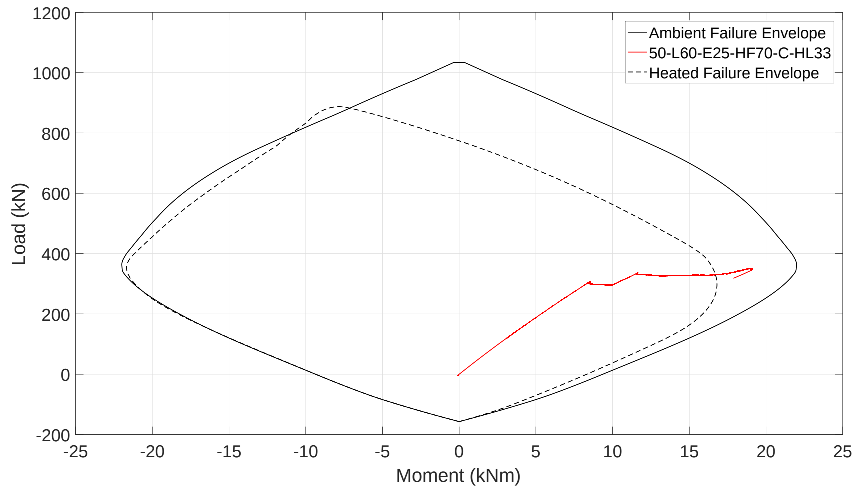


Figure 6.16: Comparison of the experimental results with the predictions of the elevated temperature sectional analysis model developed for column 50-L60-E25-HF70-C-33

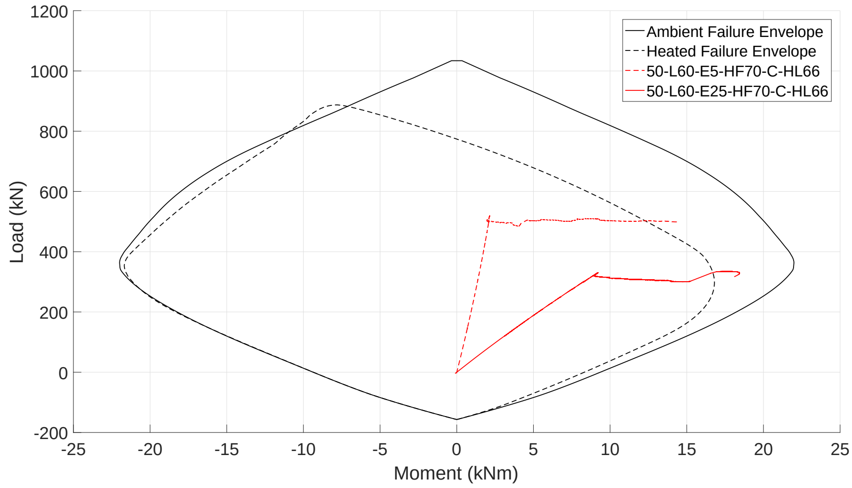


Figure 6.17: Comparison of the experimental results with the predictions of the elevated temperature sectional analysis model developed for columns 50-L60-E25-HF70-C-66 and 50-L60-E5-HF70-C-66

From the figures detailed it can be seen that the comparison of the experimental results are not as favourable for the experiments that failed in the cooling phase as it is for the experiments that failed in the heating phase of the experiments. However, of the comparisons made, the point of failure was either predicted or conservatively underestimated, as is expected from the one dimensional temperature gradient assumed and the linear interpolation of the temperatures between thermocouples. This is true for all of the comparisons in the figures detailed. It should however be noted that experiment 50-L60-E5-HF70-C-HL66, the most accurate prediction made, despite officially failing in the cooling phase of the experiment, failed when it was essentially still at its maximum temperature, 5 minutes after the heat source was removed. This is unlike the other columns in Figures 6.15, 6.16 and 6.17, which all failed roughly 2.5 hours after the heat source was removed.

It is also clear that experiments 30-L60-E25-HF70-C-33-(1) and 30-L60-E25-HF70-C-33-(2) (Figure 6.15) both achieved a comparable strength to the predicted ambient strength of the concrete columns. Although it should be noted from Figure 6.10 that the ambient strength of the C30 concrete columns was also underestimated by 8% - 15%.

6.5.5 Columns Tested Residually

Figures 6.18, 6.19, 6.20, 6.21, 6.22, 6.23 and 6.24 detail comparisons of the model developed and the actual experimental data collected for the reinforced concrete columns that were tested residually twenty-four hours after being exposed to elevated temperatures.

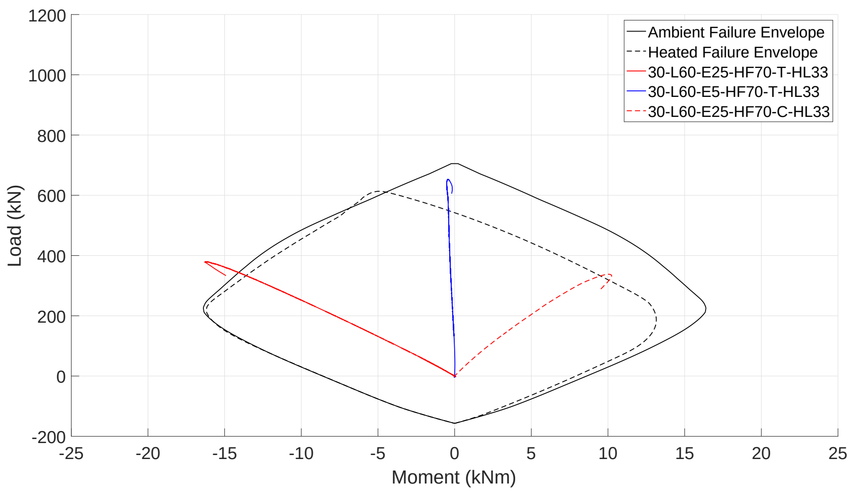


Figure 6.18: Comparison of the experimental results with the predictions of the elevated temperature sectional analysis model developed for columns 30-L60-E25-HF70-T-HL33, 30-L60-E5-HF70-T-HL33 and 30-L60-E25-HF70-C-HL33

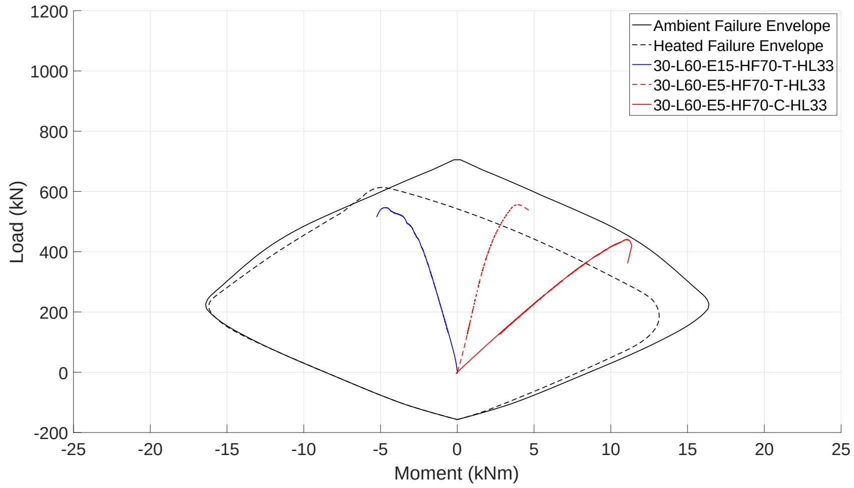


Figure 6.19: Comparison of the experimental results with the predictions of the elevated temperature sectional analysis model developed for columns 30-L60-E15-HF70-T-HL33, 30-L60-E5-HF70-T-HL33 and 30-L60-E5-HF70-C-HL33

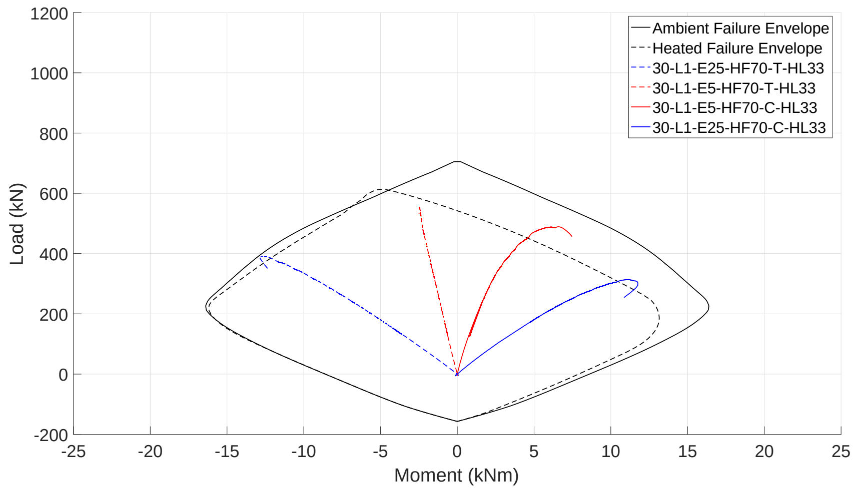


Figure 6.20: Comparison of the experimental results with the predictions of the elevated temperature sectional analysis model developed for columns 30-L1-E5-HF70-C-HL33, 30-L1-E5-HF70-T-HL33, 30-L1-E25-HF70-C-HL33 and 30-L1-E25-HF70-T-HL33

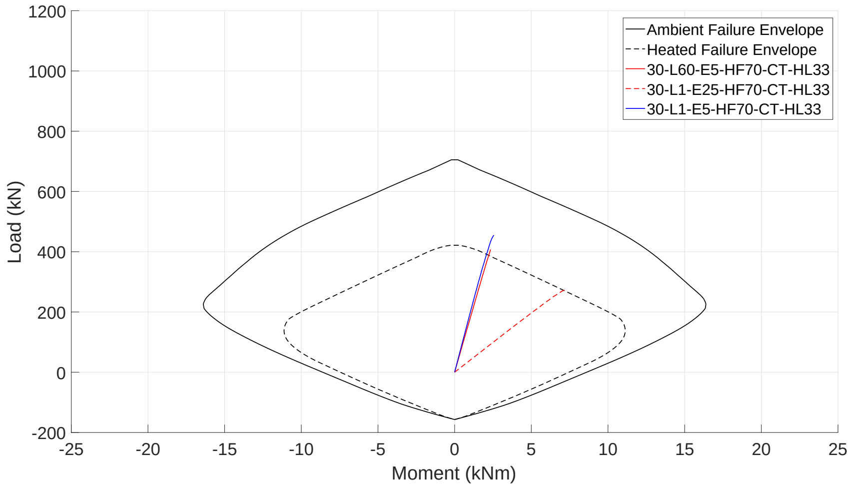


Figure 6.21: Comparison of the experimental results with the predictions of the elevated temperature sectional analysis model developed for columns 30-L60-E5-HF70-CT-HL33, 30-L1-E25-HF70-CT-HL33 and 30-L1-E5-HF70-CT-HL33

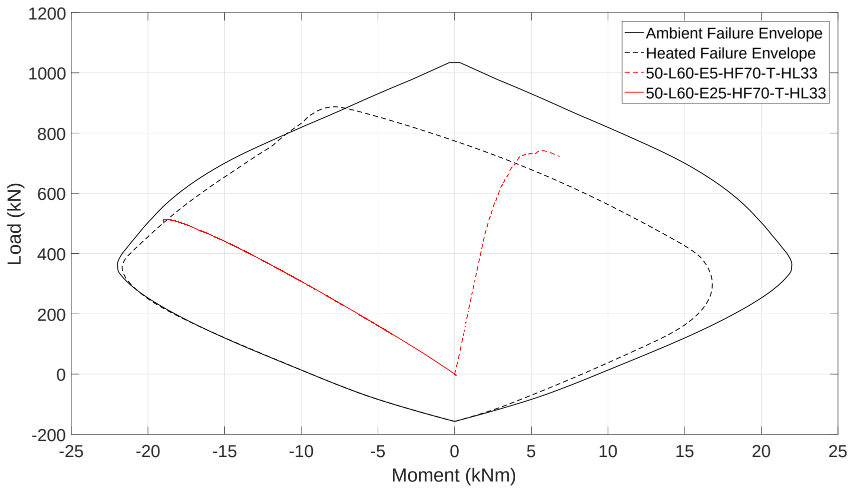


Figure 6.22: Comparison of the experimental results with the predictions of the elevated temperature sectional analysis model developed for columns 50-L60-E25-HF70-T-HL33 and 50-L60-E5-HF70-T-HL33

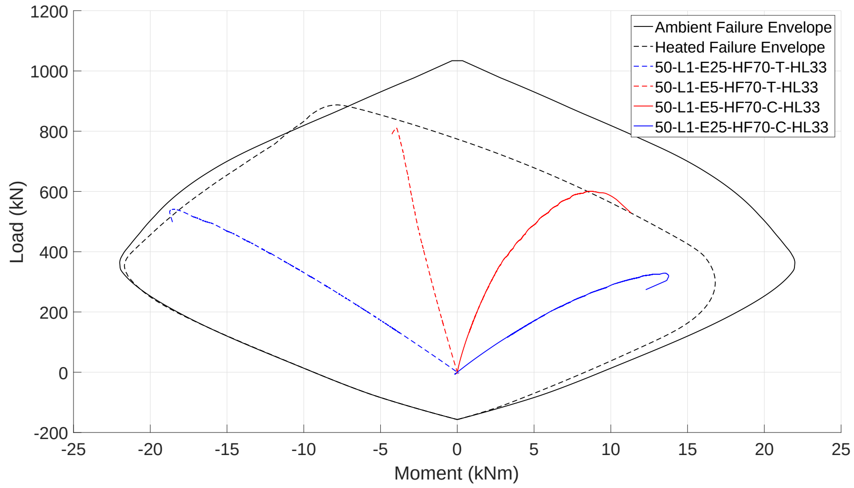


Figure 6.23: Comparison of the experimental results with the predictions of the elevated temperature sectional analysis model developed for columns 50-L1-E25-HF70-C-HL33, 50-L1-E5-HF70-C-HL33, 50-L1-E5-HF70-T-HL33 and 50-L1-E25-HF70-T-HL33

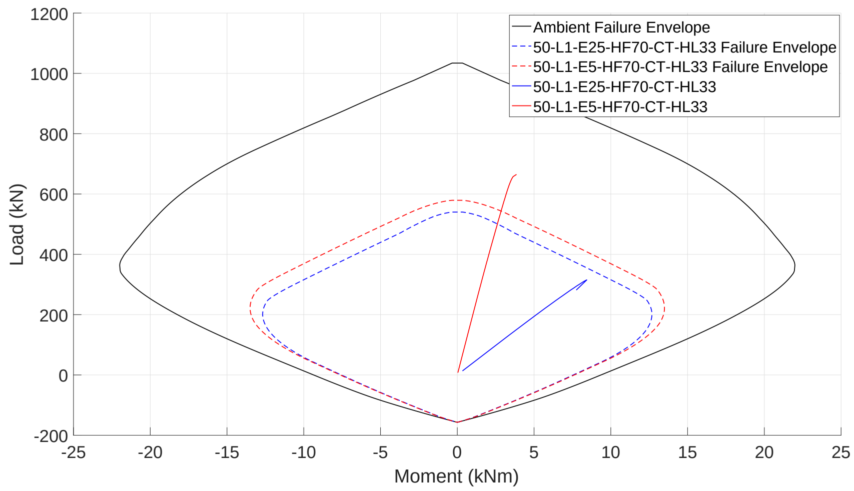


Figure 6.24: Comparison of the experimental results with the predictions of the elevated temperature sectional analysis model developed for column numbers 50-L1-E25-HF70-CT-HL33 and 50-L1-E5-HF70-CT-HL33

The figures detailed, illustrate the results of most of the experiments carried out as part of this experimental test series. In most cases, the prediction of the point of failure of the columns in the "compression heated" case (positive moment on the figures) is relatively accurate. This is also true for the point of failure of all of the experiments that were "tension heated" (negative moment on the figures). In the case of the "tension heated" columns, most of the columns achieve a comparable strength to what would be predicted by the model for the same column at ambient temperature before being exposed to elevated temperatures.

Considering the model in the ambient condition, much of the concrete on the "tension face" is ignored in the calculation due to the assumption that concrete subjected to tension does not contribute to the load bearing capacity of the element. Therefore, when exposed to elevated temperatures, much of the heated concrete that experiences a reduction in strength is subjected to tensile forces. Therefore, the remaining concrete subjected to compression forces is the source of the reduced prediction. This explanation may provide some insight into the reasons for the failure envelope falling in almost the same location as the ambient failure envelope in the "tension heated" case.

6.6 Comparison with 500^{o}C Isotherm method

In order to extend the applicability of the classic model to concrete sections exposed to elevated temperatures. The 500^{o}C Isotherm Method is recommended in Annex B of BS EN 1992-1-2 [2]. This method quite simply involves implementing the classic model detailed in Section 6.2 with a reduced cross sectional area of concrete. It is assumed that all concrete above a temperature of 500^{o}C contributes nothing to the load bearing capacity of the section. Whereas concrete below a temperature of 500^{o}C retains its ambient temperature mechanical properties [2].

The 500^{o}C Isotherm Method has therefore been implemented into the model produced in such a way that a similar approach is taken to calculate the interaction of the section. However, all concrete at a temperature of greater than 500^{o}C is ignored and all concrete under 500^{o}C is assumed to retain its full ambient temperature material properties. This approach has then been compared to the elevated temperature sectional analysis model developed earlier in this thesis, which takes account of the more precise properties of concrete at elevated temperatures, to determine how the two approaches compare in predicting the M-N interaction diagrams of the columns during and after exposure to elevated temperature. Figure 6.25 shows a comparison between the calculated M-N envelopes for the elevated temperature sectional analysis model developed as part of this research project and the 500^{o}C Isotherm Method.

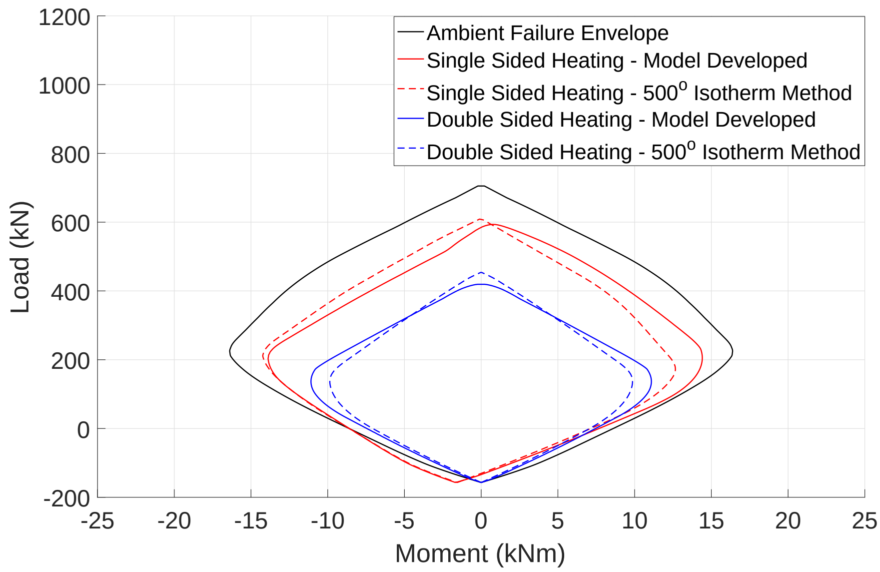


Figure 6.25: Comparison of the 500^o Isotherm Method and the elevated temperature model developed for the C30 concrete columns

A direct comparison of the two models shows that, for single sided heating, both predict a comparable reduction in strength of the columns when a tension (negative) force is applied. Within the positive moment region of Figure 6.25, it can be seen that the 500^{o}C predicts a 17% greater reduction in peak moment of the column than the model developed implementing the material properties of concrete at elevated temperature. When considering a column subjected to heating on both sides, the 500^o Isotherm Method is again more 10% more conservative than the model developed as part of this experimental test series.

Looking at the negative bending moment region of the graph, the models predict similar behaviours with the model developed estimating a 4% smaller peak moment. This section of Figure 6.25 simulates the "tension heated" scenario in the experimental test series conducted. The damaged concrete which is ignored in the 500^{o}C Isotherm Method but taken into account in the model developed is largely dismissed in both models due to the fact that it is subjected to tension forces. The remaining concrete in the section that is below 500^{o}C is stronger in the 500^{o}C Isotherm Method than in the model developed due to the fact that the 500^{o}C Isotherm Method assumes ambient conditions below 500^{o}C. Therefore resulting in a slightly greater prediction in the strength of the columns in the "tension heated" scenario. It should however be noted that this slight increase in strength is not enough to bring the 500^{o}C Isotherm Method in line with the real data collected and detailed in Section 6.5.5. The strength of the columns tested as part of this experimental test series are still underestimated by the 500^{o}C Isotherm Method.

Where the models diverge slightly further is in the calculation of the resistance of the section under a positive moment. It can be seen from Figure 6.25 that the 500^{o}C Isotherm Method predicts a 17% smaller peak moment than the model developed using the material properties at high temperature. This is due to the fact that, in the 500^{o}C Isotherm Method, the concrete above 500^{o}C is ignored and the remaining concrete in the section retains its ambient properties. Whereas the model developed takes the full spectrum of concrete material properties into account at different temperatures. From this figure it can be seen that, as the concrete increases in temperature, the strength of the concrete decreases and the ultimate stain of the concrete increases. As the positive region simulates the "compression heated" scenario from the experiments, much of the concrete that has been heated to a temperature of greater that 500^{o}C is being ignored by the 500^{o}C Isotherm Model whereas the other model takes account of the strength of the concrete above this temperature. Therefore increasing the strain on the "compression face" significantly more than on the "tension face" results in more concrete is being utilised and predicting a larger capacity in the "compression heated" scenario when taking account of the material properties of concrete at elevated temperature.

When considering the calculation of the failure envelope for the sections heated on both sides, it can be seen that the model developed predicts a slightly higher strength than the 500^{o}C Isotherm Method in both cases of positive and negative bending moments. This is due to the fact that in both the positive and negative regions, the sections remain "compression heated" as both sides are exposed to elevated temperatures, therefore resulting in a more conservative prediction of the failure envelope of the section.

6.7 Reinforced Concrete Column Failure Mechanism

Section 6.5 has discussed the comparison of the sectional analysis model developed with the results of the experiments conducted. As discussed, the model developed shows potential in predicting the M-N interaction envelope, especially when predicting the load-moment combination at the point of failure when the columns have been heated on the "compression face". Another aspect of the failure of the columns that has however been uncovered as part of this analysis, and which has profound implications for design, is the load path taken by many of the columns that failed during the heating and cooling phases of the experiment.

It has previously been discussed in Chapter 2 that the process of determining failure of a reinforced concrete column exposed to elevated temperatures can be based on the sectional properties of the concrete section. Lateral deflections are not explicitly accounted for. Many of the graphs presented in this chapter show that this is a potentially dangerous flaw in the design process.

When designing using the sectional properties of the concrete column it is only possible to determine the load-moment combination that will cause failure. In the case of an interaction diagram it is therefore assumed that, during and after a fire, the envelope of the section's interaction diagram will become smaller and approach the load-moment combination the column is subjected to. When the envelope passes below this point, the column fails. This has been illustrated in Figure 6.26.

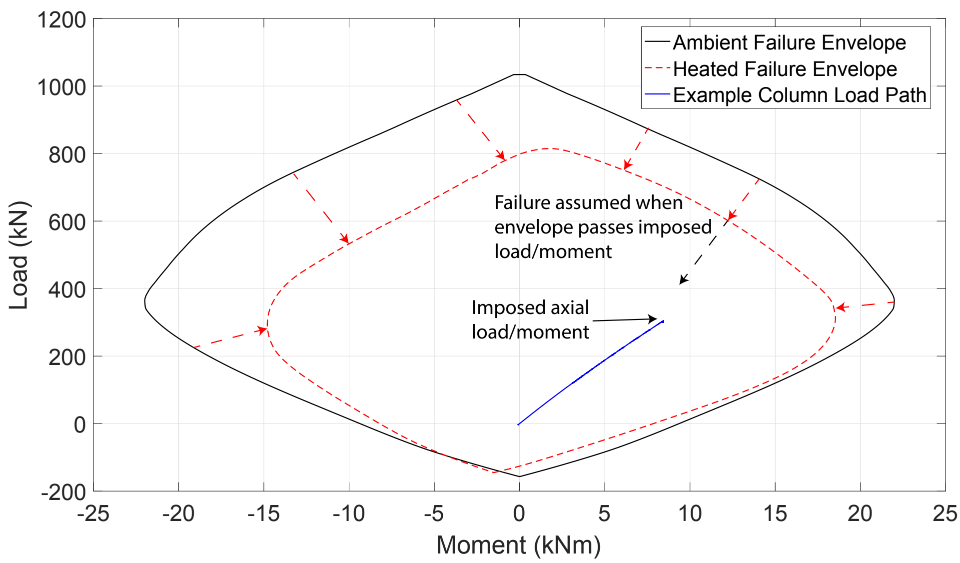


Figure 6.26: Assumed failure mechanism in sectional analysis

The results of the experimental test series conducted however very clearly illustrate that the failure mechanism assumed above is not a valid assumption for many reinforced concrete columns. As a result of the lateral deformation of the reinforced concrete columns during heating and cooling, secondary moments are induced in the section. This changes the location of the column on the load-moment interaction diagram by increasing the moment on the column. Therefore, in addition to the interaction envelope of the column reducing in size, the load-moment combination induced in the column becomes greater. It is clear from many of the graphs presented that the secondary moments induced were the defining factor that resulted in the columns failing during the heating or cooling phase of the experiment. This process has been illustrated in Figure 6.27, which details the analysis of experiment 50-L60-E25-HF70-C-33.

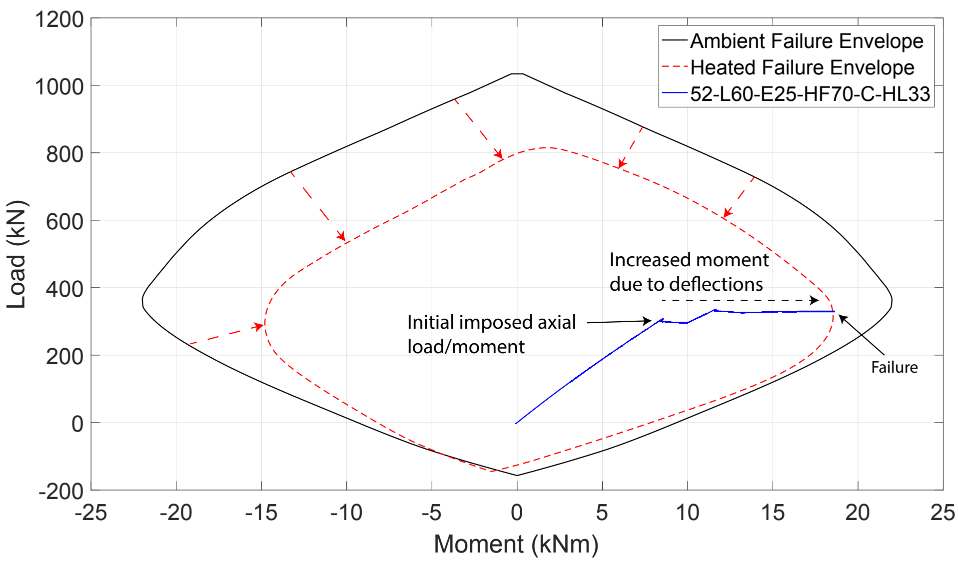


Figure 6.27: Actual failure mechanism in experiments conducted

Figure 6.27 clearly illustrates that, even given the reduced M-N envelope of the section, there remains a significant factor of safety when considering the initial load conditions that the column was subjected to. Despite this apparent factor of safety, the column failed as a result of the moment in the column doubling during the heating phase of the experiment as a result of the lateral deflections induced.

This illustrates the fact that more robust design techniques are required in the analysis of reinforced concrete columns subjected to elevated temperatures. Any analysis that defines condition of a reinforced concrete column subjected to elevated temperatures is required to take account of the secondary effects induced by the deflection of the columns. This necessary will require the understanding and inclusion of concrete specific phenomena such as transient thermal creep into any model created.

6.8 Chapter Conclusions

Throughout Section 6, two possible approaches in determining the residual strength of reinforced concrete columns after exposure to elevated temperatures has been presented. This is currently an area which is not covered in great detail within BS EN 1992-1-2. Where the calculation of the capacity of a section exposed to elevated temperatures is required, tabulated data or simplified calculation methods are recommended (such as the 500^{o}C Isotherm Method). Without the use of more advanced calculation methods, there is currently no guidance on how to more accurately account for the varying material properties through a section resulting from exposure to elevated temperatures.

A sectional analysis model has been developed and compared against the actual experimental test data gathered using both the material models recommended in BS EN 1992-1-2 and the 500^{o}C Isotherm Method. Varied degrees of success have been achieved in this comparison. From Section 6.5, it is clear that at ambient temperature, the procedure followed predicts the points of failure of the columns within 10% for the weaker concrete columns and within 1% for the stronger concrete columns. This was however, to be expected, as the similar classical approach detailed in Section 6.2 has been used for many years now in design. The results of the model do however become significant when comparing the M-N enveloped of the heated columns to the actual experimental data collected. The section analysis model developed has varying degrees of success in predicting the point of failure of the columns that failed during all stages of the experiment (heating, cooling, and residually).

Throughout Sections 6.5.3 - 6.5.5, the predicted failure envelope for the columns based upon the temperature gradient within the section has been detailed, along with the actual data for 29 experiments. Of these comparisons, the failure envelope overestimated the point of failure of just five of the columns by around 6% in each case. The failure of the remaining columns was either accurately predicted or underestimated. In addition to this, the model developed was campared with the 500^{o}C Isotherm Method recommended within BS EN 1992-1-2. It was concluded from this comparison that the 500^{o}C Isotherm Method is more conservative (by as much as 15% in this scenario) than the model developed and would under predict the point of failure of every single column tested as part of the experimental test series.

It is therefore clear from the results of the comparisons made that the 500^{o}C Isotherm Method may be a "safe" method to use in predicting the residual strength of a reinforced concrete section. However, as the failure point of every experiment conducted would be underestimated, one could argue that it is perhaps too conservative a method, leading to over design of sections. The model developed however, more reliably predicts the exact point of failure of the columns tested whilst still remaining conservative in the prediction of the experiments it can not accurately reproduce.

6.9 Applicability to Design and Limitations

Throughout Chapter 6, a sectional analysis model has been presented. This has shown that it is possible to predict the residual strength of a reinforced concrete column after exposure to elevated temperatures relatively accurately. It should however be noted that, the tests conducted were done so under controlled conditions with all of the critical measurements required for the analysis continuously measured throughout the experiments. This means that the predictions made can be confidently compared to the experimental results however, looking at a potential fire in a real building, the situation is considerably less favourable. In order to predict the failure envelope of a concrete section after fire using the model presented (or any model currently available for that matter), exact details on the following parameters are required:

- the maximum temperature achieved through the depth of the section;
- the material properties of the concrete and steel after exposure to elevated temperatures, and;
- the residual displacements of the element.

Maximum Temperature Achieved Through the Depth of the Section

In order to gather this information, thermocouples were cast into the concrete during the experiments conducted. In a real building, no such instrumentation is available, making the temperature gradient through the column a difficult parameter to estimate. Current approaches to estimate this information involve simulating a likely fire and conducting a heat transfer analysis through the section based on the exposure temperature. There are a number of uncertainties in undertaking this process however, most of all in the simulation of the fire itself.

It may however be possible to estimate the temperature gradient in some specific cases, as discussed in Chapter 5. It was possible, using a non-destructive testing technique to relatively accurately determine the temperature gradient achieved through the sections. This may therefore allow engineers to estimate the failure envelope of the concrete section using the model predicted however, to determine the load path of a section, the residual displacements are also required along with accurate material properties.

Material Properties

In conducting this experimental test series, the material properties of the concrete used were not investigated thoroughly at elevated temperature. Instead, the material properties of concrete and steel recommended in BS EN 1992-1-2 were used. This allowed a relatively accurate prediction of the failure envelope of the columns. It would however be beneficial in any real investigation carried out to have a better idea of the real properties of the concrete. Something that, in a building constructed 20 years previously, is unlikely to be available. Depending on the records, the ambient strength of the concrete may not even be available. In these cases, it may only be possible to gather this information by taking cores of undamaged concrete from the building itself. A costly, time consuming, and semi-destructive technique.

Residual Displacements

The residual displacements, like the temperature gradient, were measured through the course of the experiments. They are however, much like the temperature gradient, unavailable in the analysis of a "real" structural element. It may be possible to estimate the displacements based upon the residual curvature of an element but this approach is likely to have a number of errors inherent within, and would assume that the structural element was straight to begin with. In addition, the surrounding structure may have redistributed the load within the building and changed the eccentricity of the load through the course of the fire. In order to fully determine the conditions of a structural element after a fire and the probable load path that it will follow on the interaction diagram predicted, this information is required.

Therefore it can be concluded from Chapter 6 that the it may be possible to accurately determine the residual failure envelope of a reinforced concrete column during, or after exposure to elevated temperatures. In certain circumstances this may even be possible without detailed data on the temperature gradient through the structural element. As discussed in Chapter 5, it may be possible to determine this retrospectively. However, to apply these predictions to a real building, additional techniques in determining the real displacements and loads on the elements are required. Until these can be accurately resolved, this method may only provide engineers with an indicative reduction in strength of a concrete section. A useful tool to help inform the post-fire reparations, but by no means a final answer to the question of post-fire damage to concrete structures.

Chapter 7

Conclusions and Recommendations

Throughout this thesis, the results of a series of experiments conducted on 46 reinforced concrete columns have been presented and discussed. These concrete columns were axially loaded in eccentric compression and subjected to a localised heat flux of varying intensity to investigate the effects of these parameters on the structural performance of reinforced concrete columns both during and after a fire. Experimental data were gathered before heating in the ambient temperature condition, during heating when the columns were subjected to elevated temperatures, and after heating when the columns were in the cooling phase to capture the structural response of the elements throughout the full process of being exposed to elevated temperatures. The columns that remained structurally viable after cooling back to ambient temperature conditions were destructively tested 24 hours after the load was first applied, to determine their post-heating residual structural response and capacity. The current design guidance used by engineers in the design and construction of reinforced concrete columns has been compared to the data gathered as part of this experimental test series, to determine current gaps in knowledge and guide future work in this area of research (and in practice).

7.1 Implications for Design

A large volume of experimental data and analysis have been presented within this thesis, much of which has concentrated on determining the effect of numerous loading and heating conditions on the response of one specific reinforced concrete column. Some real-life context has however been provided for this in Section 4.11.

It should be noted that all of the external parameters investigated as part of this experimental test series affect one of the two parameters which ultimately determines the response of the column; the stress distribution within the section, and the temperature gradient through the section. These two parameters will ultimately determine the material behaviour of the constituent parts of the columns. For a full discussion on how these parameters affect the response, and the importance of fully understanding them in quantitative terms, see Chapter 6. A brief discussion and summary of the conclusions reached during this work is provided in the following sections.

7.1.1 Heating and Cooling Phase

Several conclusions have already been presented throughout the thesis as a result of the behaviour of the reinforced concrete columns in the heating and cooling phases of the experiments. Most importantly, it must be noted that every parameter investigated has the potential to have a considerable effect on the structural response of the columns during heating, cooling, or residually. In the structural fire design of reinforced concrete columns in practice, the guidance provided does not necessarily (indeed very rarely, if ever) explicitly considers all of the parameters investigated in this thesis and would assume an identical response subject to some variance in essentially all of them.

The Eurocode suite of guidance documents [2, 4, 12, 93, 94] suggests finding the temperature gradient through the concrete section before conducting a structural analysis. When calculating the temperature gradient through the section, however, no details are provided on how to determine the specific heating conditions that might be experienced in a real fire. Instead, it is recommended that the standard fire curve, the so-called parametric fires [93], or similar standardised heating curves are used. The general assumption behind implementing this method in practice (although not its original intent) seems to be that this is a credible worst case conservative fire curve. The idea is apparently that this will result in redundancy against fire loading of the element being designed. Although this may be true in many cases during the heating phase of a fire where, the higher/longer heat flux will result in greater damage and deflections, this is not necessarily true when considering the residual response of the columns (which is not required to be explicitly considered at all in any guidance document for design), see Section 7.1.2.

When completing a structural analysis of the reinforced concrete columns, a zoned model [2], 500^{o}C isotherm method, or a more detailed sectional analysis structural analysis is generally recommended. For a comparison of these candidate methods when compared against the experimental data generated within this project, see Chapter 6.

Nowhere within the available guidance is it suggested that transient thermal creep of concrete, which is known to occur under load at high temperature, is to be explicitly considered in any analysis conducted (however many authors have suggested, with scant evidence or justification in most cases, that it is implicitly included in material models for concrete applicable to standard heating curves). When performing sectional analysis of concrete columns at elevated temperature, this means that practitioners are able to determine the cross-sectional capacity of a column subjected to certain axial load/moment combinations during fire via calculation of the axial-flexural interaction diagram.

However, it is not possible by this approach to determine the lateral deflections of the columns during heating or cooling; this parameter can ultimately determine the severity of the moments induced within the column. Therefore, in any analysis undertaken on a reinforced concrete column subject to axial stresses large enough to induce transient thermal creep within the section during heating, a standard high temperature constitutive model for concrete, considering only the reduction in strength of the column and not the additional secondary moments induced by increased lateral deflections from thermal bowing or transient thermal creep, may ultimately overestimate the fire resistance of the element, or may result in an element which achieves the required fire resistance but fails during the cooling phase of a fire. This raises particular concerns with regard to any fire engineered solutions which may rely on such an analysis to omit the need for normal fire resistance requirements on, for example, a slender, high strength, reinforced concrete column.

Looking specifically at the cooling phase of the experiments, it has been demonstrated (in Section 4.6) that after the heat flux has been removed i.e. the fire has been extinguished, a reinforced concrete column may still sustain considerable additional lateral deformation during the cooling phase. During the cooling phase of the experiments, it was observed that concrete under high compressive stresses that experienced transient thermal creep during heating, continued to contract further during the cooling phase, to the point where midspan deflections could double in the hours after the heat source has been removed.

In practical design of reinforced concrete buildings, the cooling phase of a fire is not generally explicitly considered or analysed by designers, due to the assumption that, if the column maintains structural integrity for the defined period of fire resistance time, it can be assumed to be adequately safe. However this says nothing explicit about firefighter safety, fire investigator safety, building resilience, or repair and re-use of a structure.

In addition to this potential flaw in the structural fire resistance design process, it could be argued that the assumption of safety upon achieving a certain fire resistance period, considered by many to be due to the fact that the occupants should have evacuated safely after the fire resistance time has been met, is entirely inappropriate. This is particularly the case for critical structural elements (i.e. load-bearing columns) in a staged or phased evacuation scenario where cooling of structural elements is within the domain of the design. Should the cooling phase be ignored due to the fact that it is not important from a life-safety perspective, a failure of a column in the cooling phase, unlike other types of structural element, could result in collapse of a portion of structure. Should additional columns, where the load is redistributed also be in a post-fire cooling phase, full structural collapse could be possible. Obviously, these ideas need to be interrogated in future research using finite element models that can now be properly validated based on their ability to predict the results of the experiments presented in this thesis. The cooling phase is therefore not only of concern to those within the building, but also for the persons and buildings in the immediately surrounding area.

7.1.2 Residual Performance After Cooling

Structural Performance

From a post-fire residual structural performance standpoint, there have been a number of questions raised as part of this experimental test series regarding how practitioners can approach this type of assessment and analysis. The general conclusion from Chapter 5 is that, if the peak temperatures experienced within the section can be accurately estimated, it may be possible to estimate the residual strength of that element, in the form of a post-fire axial-flexural interaction diagram. It has been concluded from this analysis that both the classical approach of calculating the interaction diagram of a section and the model developed by the author both produce conservative results when compared against the test data obtained during the current project.

The model developed by the author predicted the point of failure more accurately than the classical approach. However, it is clear that further work is required in determining the residual performance of concrete, especially in the case of a tension heated column, the post-heating residual capacity of which was consistently underestimated by the analysis approaches considered herein. It could be argued that this is fine since the predictions are conservative, however it highlights that there are additional mechanics at play that are not fully understood at the time of writing.

The second caveat in the use of this approach, and potentially the most difficult to address, is that failure of a column in reality results from a combination of load and moment - and the moments fundamentally depend on the lateral deflections of the column induced during heating (and potentially also during cooling). The analysis in Chapter 6 is capable of calculating every combination of load and moment that will result in failure of the column It is not however capable of predicting what specific combination of load and moment will result in failure of a specific column loaded to a specific eccentricity with a degree of residual lateral deflection. For this to be possible it is essential that the residual deformations in the structure are known as this will determine exactly what load path on the interaction diagram the column is likely to follow.

As a result of the residual lateral deformations in the column, this load path will not be the same as the load path taken by the columns in the original ambient temperature condition experiments. As it is difficult to accurately determine the residual deflections of a column and the applied load, which may be larger than in the ambient temperature condition due to possible load redistributions, this method is not currently effective in predicting the post-fire structural response of a reinforced concrete columns exposed to elevated temperatures.

Further work is therefore required in predicting not only the residual strength of reinforced concrete columns, but more importantly the load path that the columns are likely to take in the residual state after a fire. This type of analysis is the one that will provide designers and practitioners confidence when performing a residual damage assessment. The interaction diagram approach proposed in Chapter 6 may be useful as an additional tool but cannot and should not be used in isolation to determine the residual performance of a reinforced concrete column after exposure to elevated temperatures.

The data provided in this thesis are, to the knowledge of the author, the only data currently available within the research literature that can be used to carefully validate the ability of finite element models for concrete at elevated temperature; this is particularly the case for reinforced concrete columns in which transient thermal creep (and possibly other factors) is likely to play a significant role in influencing lateral deformations during both heating and cooling.

Non-Destructive Testing for Post-Fire Damage to Concrete

In testing the Ultrasonic Pulse Velocity through the reinforced concrete columns, it was possible to estimate the maximum temperature achieved at each depth within the concrete column. This is a promising result from a residual analysis perspective because a presents a step forward in that it was possible to determine, within a small margin of error, the maximum temperature achieved at each point through the depth of the columns. However, it should be noted that the conditions tested were ideally suited for such results (i.e. effectively one-dimensional) heat transfer with easy access to all four sides of the columns. These conditions will not be present in most scenarios due to access difficulties of different structural arrangements and non-ideal heat exposure (more than single side heating, or non-uniform heating with unknown distributions); this introduces many more uncertainties to the application of this technique.

It is however clear that, in certain scenarios, Ultrasonic Pulse Velocity may be utilised to determine the extent of damage to a concrete element as a result of exposure to elevated temperatures. This confirmation opens doors to different types of analysis and ways forward to improve the approach. The method of data capture used in the experimental test series presented is that of a hand-held Pundit that required the user to collect data points one at a time. This resulted in difficulty when collecting data over a larger area due to the labour costs. In more complicated scenarios it may be possible to take a more automated approach to the data capture to allow for a higher resolution of data points on all faces of the concrete. It may then be possible to analyse this data map to produce a more in-depth damage model of the element at much lower time and labour costs. This type of approach may also solve the issues stated earlier regarding access and more complex heat transfer scenarios.

7.1.3 Experimental Limitations

An area of the experimental test series that may have been improved is the load applied to the columns during each experiment. The load applied to the columns was calculated as a percentage of the total capacity of the column under a certain loading condition. As the capacity of the columns differs between load eccentricity and the strength of the concrete (two of the variables investigated), the load in the columns and the stress in the concrete differs for each of the experiments. This makes direct comparison between experiments difficult when comparing load eccentricity and concrete strength. All of data required to calculate stresses and strains in the columns have however been collected and made available via a data repository, thus allowing for some comparison between columns of different concrete strengths and for validation of future modelling campaigns. Future experimental test series may benefit from applying the load on the columns with the intention of inducing a similar level of compressive stress in the columns.

7.2 Conclusions

From the research presented as part of this thesis, the primary conclusions that can be taken from the work are:

- There is currently a paucity of data available on the performance of reinforced concrete columns subjected to elevated temperatures (particularly for 'non-standard' heating scenarios). Much of the work to date has been conducted in standard fire resistance testing furnaces or other experimental test setups that make careful data acquisition a challenging task. It is therefore difficult when modelling these experiments to make comparisons with, for example, midspan lateral deflections and other physical parameters. It is therefore not possible to thoroughly validate the most advanced finite element models developed as internal temperatures and "time to failure" are the only parameters that can be carefully compared against the experimental data in most cases.

- Reinforced concrete columns subjected to elevated temperatures behave in a significantly different manner when exposed on the "compression face" compared to the "tension face". In the work presented this is attributed to the differential compressive stress in the columns on the "tension face" and the "compression face". It was observed that, as the level of compression on the exposed face increased, the lateral deflections increased, thus decreasing the effective "fire resistance" of the column specifically because of the secondary moments induced by these additional lateral deflections. This has profoundly important implications for the current design processes that are used for fire resistance design of reinforced concrete columns; these do not typically consider secondary effects in calculating the "fire resistance" of the element. In the work presented herein, the secondary effects induced by the deflections were the deciding factor on whether a column would fail during heating, fail during cooling, or survive the exposure until returning to cold conditions (and then what post-heating residual capacity is retained).

- When conducting a post-fire condition assessment of a reinforced concrete structure, the use of Ultrasonic Pulse Velocity is able to provide an accurate 'map' of the maximum temperatures experienced within the element. Additional work is required to refine the technique to the point that it is capable of being deployed as an effective tool for post-fire assessment. However, in the work presented herein, it has proven to be a promising tool that warrants further investigation for such scenarios.

- The current analytical methods used in the development of M-N interaction diagrams for a reinforced concrete column can be used to provide a conservative estimate of the strength of a column when applied to a column at elevated (and after) temperatures. An additional model based upon a 'zone model' approach has been presented that provided a more accurate estimate for the reinforced concrete columns when exposed to elevated temperatures on the "compression face". However, the model presented still considerably underestimated the strength of a reinforced concrete column exposed to elevated temperatures on the "tension face" by as much as 20% in certain instances.
- A critical factor in determining the structural fire response and capacity of a reinforced concrete column subjected to a non-uniform heating and cooling regime is the secondary effects of the deflections induced. The current analytical design methods only consider the reduction in the strength of the column cross section. Ultimately, failure of the reinforced concrete columns in this experimental test series was caused by the changing load conditions (increase in moment), which resulted from the increase in the midspan deflections. Therefore, when considering the M-N interaction diagram of the columns, failure is caused by the load condition increasing and approaching the failure envelope, rather than only the reduction in the size of the envelope because of the weakening on the concrete.

7.3 Recommendations

In the author's opinion, the current guidance in the design and construction of reinforced concrete columns offers a good estimation of the strength of an element when subjected to a specific load case during and and after exposure to elevated temperatures. What is however less clear, is the ability of any practitioner to accurately determine the load case beyond first exposure to heat, especially when transient thermal creep is likely to be involved, which is the case for all compressive concrete (and some flexural) elements.

Chapter 6 details how the failure envelope for a given cross-section can be estimated, however the deflections of the columns are a much more challenging to determine (and arguably more important to understand). Therefore, before completing such an analysis, more work is required in understanding the effect that different load/heating scenarios have on the extent of transient thermal creep experienced within the concrete columns and how this affects the displacements experienced.

A full Finite Element analysis of the columns used in the experiments would be a very beneficial investigation to undertake. In conducting the column experiments as part of this thesis, a large and unique pool of data on the performance of concrete during and after exposure to elevated temperatures has been collected. This provides structural fire engineering modellers with the ability to compare and contrast the current understanding with a large and previously unavailable data resource on interesting and potentially important thermal and mechanical responses of reinforced concrete columns under axial-flexural loading. Such an analysis would expose the gaps in knowledge of the response of reinforced concrete columns at elevated temperatures.

An area of investigation that shows promise in the area of post-fire damage analysis of concrete structures is that of the Ultrasonic Pulse Velocity technique described in Chapter 5. As previously described, there are a number of caveats when considering the application of this technical in the form it has been applied as part of the experimental test series described. There are however more advanced data capture methods available in this area that could not be utilised as part of this thesis. The use of these more advanced methods of data acquisition may allow for more in depth analysis of the columns an production of 3D models to visualise the damage caused by exposure to elevated temperatures. Further investigation into the application of Ultrasonic Pulse Velocity in the determination of damage to concrete structures may provide engineers with a valuable tool to be utilised for post-fire damage assessments.

In reviewing the literature available on the performance of reinforced concrete columns during and after a fire, it is clear that there is currently a deficit of good data available for complex analysis models to be validated against. This is in part due to a lack of experimental test series being conducted on this type of element. However, many test series that have been conducted have collected a sparsity of data due to difficulties in doing so for the type of tests conducted. Fire Engineering as a discipline has very well-defined approaches for determining the "fire resistance" of different elements and materials. In Structural Fire Engineering these are known as the "standard fire curves" and "fire resistance ratings", an essential suit of temperature time curves that allow practitioners to conduct standardised tests to compare "like-for-like". This does not however imply that the standard fire curves are suitable in determining the response of a material/element/structure to fire in every credible worst-case fire scenario. In the author's opinion, this is a mentality that significantly hinders innovation within Fire Engineering.

There is an almost infinite number of scenarios in which a structure can be exposed to elevated temperatures, it is naive and irresponsible as a community to expect advances when the same tests are performed repeatedly. The expense of these tests and the resources required also prevent experiments from being conducted because, in many cases, that volume of money cannot be spared. As opposed to relying on furnace tests following a standard fire curve (which are inherently difficult to instrument and provide limited data compared to the resources required), researchers should be innovating with different types of exposure and data capture to help stimulate progress and the development of cheaper, more flexible standardised testing regimes. Only at this point will Fire Engineering as an industry step beyond its dependency on non-physical parameters to describe "fire resistance", to being able to confidently predict structural behaviour during and after a real fire and therefore to credibly and demonstrably quantify the level of fire safety delivered by any candidate design, both in terms of life safety, and in terms of property protection and infrastructure resilience.

References

[1] J.-m. Franssen, "DESIGN OF CONCRETE COLUMNS BASED ON EC2 TABULATED DATA - A CRITICAL REVIEW," no. June, pp. 323–340, 2000.

[2] BS EN 1992-1-2:2004, "Eurocode 2 : Design of concrete structures - Part 1-2: General rules - Structural fire design," *BRITISH STANDARDS INSTITUTION*.

[3] Q. Ma, R. Guo, Z. Zhao, Z. Lin, and K. He, "Mechanical properties of concrete at high temperature-A review," *Construction and Building Materials*, vol. 93, pp. 371–383, 2015.

[4] BS EN 1992-1-1:2004, "Eurocode 2 : Design of concrete structures - Part 1-1: General rules and rules for buildings," *BRITISH STANDARDS INSTITUTION*.

[5] V. Kodur, "Properties of Concrete at Elevated Temperatures," vol. 2014, 2014.

[6] G. Torelli, M. Gillie, P. Mandal, and V.-X. Tran, "A multiaxial load-induced thermal strain constitutive model for concrete," *The International Journal of Solids and Structures*, vol. 108, pp. 1–11, 2016.

[7] D. To and M. A. Difference, "The Cement Sustainability Initialtive - Our

Agenda for Action," Tech. Rep. July, World Business Council for Sustainable Development, 2002.

[8] M. Ahrens, "High-Rise Building Fires," Tech. Rep. November, NFPA, 2016.

[9] J. Ingham, "Forensic engineering of fire-damaged structures," *Proceedings of the ICE - Civil Engineering*, vol. 162, pp. 12–17, jan 2009.

[10] F. Block, J. Clarke, B. Cole, S. Deeny, A. Heise, J. Ingham, and N. Pierce, "Assessment, Design and Repair of Fire-Damaged Concrete Structures." 2013.

[11] HM Government, "Approved Document B: Volume 2 - Buildings other than Dwellinghouses," 2013.

[12] BS EN 1991-1-2:2002, "Eurocode 1 : Actions on structures - Part 1-2: General actions - Actions on structures exposed to fire," *BRITISH STANDARDS INSTITUTION*.

[13] The Scottish Government, "Technical Handbook - Domestic," 2017.

[14] ASTM, "E119-16 - Standard Test Methods for Fire Tests of Building Construction and Materials," *American Society for Testing and Materials*, pp. 1–36, 2016.

[15] BSI, "BS ISO 834-10:2014 Fire resistance tests. Elements of building construction. Specific requirements to determine the contribution of applied fire protection materials to structural steel elements," 2014.

[16] J. M. Franssen, V. K. R. Kodur, and J. Mason, "User's Manual for SAFIR (Version NZ). A Computer Program for Analysis of Structures Submitted to the Fire." 2016.

[17] I.-C. Dotreppe, J.-M. Franssen, A. Bruls, R. Baus, P. Vandevelde, R. Minne, D. V. Nieuwenburgt, and H. Lambottet, "Experimental research on the

determination of the main parameters affecting the behaviour of reinforced concrete columns under fire conditions," no. 179, pp. 117–127, 1997.

[18] L. T. Phan, N. J. Carino, D. Duthinh, and E. Garboczi, "Fire test on normal and high-strength reinforced concrete columns." 1997.

[19] C. G. Bailey and G. A. Khoury, "Performance of Concrete Structures in Fire," tech. rep., The Concrete Centre.

[20] A. Pineaud, P. Pimienta, S. Rémond, and H. Carré, "Mechanical properties of high performance self-compacting concretes at room and high temperature," *Construction and Building Materials*, vol. 112, pp. 747–755, 2016.

[21] Y. Chang, Y. Chen, M. Sheu, and G. Yao, "Residual stressstrain relationship for concrete after exposure to high temperatures," *Cement and Concrete Research*, vol. 36, pp. 1999–2005, oct 2006.

[22] T. Drzymała, W. Jackiewicz-Rek, M. Tomaszewski, A. Kuś, J. Gałaj, and R. Šukys, "Effects of High Temperature on the Properties of High Performance Concrete (HPC)," *Procedia Engineering*, vol. 172, pp. 256–263, 2017.

[23] Y. Anderberg and S. Thelandersson, "Stress and deformation characteristics of concrete at high temperatures, 2-Experimental investigation and material behaviour model," 1976.

[24] R. Felicetti and P. G. Gambarova, "Effects of high temperature on the residual compressive strength of high strength siliceous concretes," *ACI Materials Journal*, vol. 95, no. 95, pp. 395–406, 1998.

[25] L. T. Phan and N. J. Carino, "Code provisions for high strength concrete strength-temperature relationship at elevated temperatures," *Materials and Structures/Materiaux et Constructions*, vol. 36, no. 256, pp. 91–98, 2003.

[26] L. T. Phan and N. J. Carino, "Review of Mechanical Properties of HSC at Elevated Temperature," *Journal of Materials in Civil Engineering*, vol. 10, no. 1, pp. 58–65, 1998.

[27] Y. F. Fu, Y. L. Wong, C. S. Poon, and C. A. Tang, "Stressstrain behaviour of high-strength concrete at elevated temperatures," *Magazine of Concrete Research*, vol. 57, no. 9, pp. 535–544, 2005.

[28] J. Xiao and G. König, "Study on concrete at high temperature in China - An overview," *Fire Safety Journal*, vol. 39, no. 1, pp. 89–103, 2004.

[29] M. Husem, "The effects of high temperature on compressive and flexural strengths of ordinary and high-performance concrete," *Fire Safety Journal*, vol. 41, pp. 155–163, mar 2006.

[30] M. Li, C. Qian, and W. Sun, "Mechanical properties of high-strength concrete after fire," *Cement and Concrete Research*, vol. 34, pp. 1001–1005, jun 2004.

[31] X. Luo, W. Sun, and S. Y. N. Chan, "Effect of heating and cooling regimes on residual strength and microstructure of normal strength and high-performance concrete," *Cement and Concrete Research*, vol. 30, pp. 379–383, mar 2000.

[32] Y. Chan, X. Luo, and W. Sun, "Compressive strength and pore structure of high-performance concrete after exposure to high temperature up to 800C," *Cement and Concrete Research*, vol. 30, no. 2, pp. 247–251, 2000.

[33] Y. Chan, G. Peng, and M. Anson, "Residual strength and pore structure of high-strength concrete and normal strength concrete after exposure to high temperatures," *Cement and Concrete Composites*, vol. 21, no. 1, pp. 23–27, 1999.

[34] C. Castillo and A. J. Durrani, "Effect of transient high temperture on high-strength concrete," *ACI Materials Journal*, vol. 87, no. 1, pp. 47–53, 1990.

[35] F. P. Cheng, V. K. Kodur, and T. Wang, "Stress-strain curves for high strength concrete at elevated temperatures," tech. rep., NRCC-46973, 2004.

[36] C. Cruz, "Elastic properties of concrete at high temperatures," *Journat of the PCA Research and Development Laboratories*, vol. 8, pp. 37–45, 1966.

[37] I. Rickard, L. Bisby, and S. Deeny, "Exposive Spalling of Concrete in Fire: Novel Testing to Mitigate Design Risk," *The Structural Engineer*, pp. 42–47, 2018.

[38] C. Maluk, "Development and Application of a Novel Test Method for Studying the Fire Behaviour of CFRP Prestressed Concrete Structural," pp. 22–30, 2014.

[39] R. Jansson, *Fire Spalling of Concrete: Theoretical and Experimental Studies (Doctoral Thesis, KTH Royal Institute of Technology).* PhD thesis, 2013.

[40] G. A. Khoury, P. J. Sullivan, and B. N. Grainger, "Transient thermal strain of concrete: literature review, conditions within specimen and behaviour of individual constituents," *Magazine of Concrete Research*, vol. 37, no. 132, pp. 131–144, 1985.

[41] G. Khoury, B. Grainger, and P. Sullivan, "Strain of concrete during first heating to 600C under load," *Magazine of Concrete Research*, vol. 37, no. 133, pp. 195–215, 1985.

[42] G. Khoury, B. Grainger, and P. Sullivan, "Strain of concrete during first cooling from 600oC under load," *Magazine of Concrete Research*, vol. 38, no. 134, pp. 3–12, 1986.

[43] G. Khoury, B. Grainger, and P. Sullivan, "Strain of concrete during first heating to 600C under load," *Magazine of Concrete Research*, vol. 37, no. 133, pp. 195–215, 1985.

[44] G. Torelli, P. Mandal, M. Gillie, and V. X. Tran, "Concrete strains under transient thermal conditions: A state-of-the-art review," *Engineering Structures*, vol. 127, pp. 172–188, 2016.

[45] F. Aslani, "Prestressed concrete thermal behaviour," *Magazine of Concrete Research*, vol. 65, no. 3, pp. 158–171, 2013.

[46] J. C. Mindeguia, I. Hager, P. Pimienta, H. Carré, and C. La Borderie, "Parametrical study of transient thermal strain of ordinary and high performance concrete," *Cement and Concrete Research*, vol. 48, pp. 40–52, 2013.

[47] G. A. Khoury, C. E. Majorana, F. Pesavento, and B. A. Schrefler, "Modelling of heated concrete," *Magazine of Concrete Research*, vol. 54, no. 2, pp. 77–101, 2002.

[48] L. Y. Li and J. Purkiss, "Stress-strain constitutive equations of concrete material at elevated temperatures," *Fire Safety Journal*, vol. 40, no. 7, pp. 669–686, 2005.

[49] C. V. Nielsen, C. J. Pearce, and N. Bicanic, "Theoretical model of high temperature effects on uniaxial concrete member under elastic restraint," *Magazine of Concrete Research*, vol. 54, no. 4, pp. 239–249, 2002.

[50] T. Gernay and J. M. Franssen, "A formulation of the Eurocode 2 concrete model at elevated temperature that includes an explicit term for transient creep," *Fire Safety Journal*, vol. 51, pp. 1–9, 2012.

[51] T. Lie and T. Lin, "Influence Of Restraint On Fire Performance Of Reinforced Concrete Columns," *Fire Safety Science*, vol. 1, pp. 291–300, 1986.

[52] F. Ali, A. Nadjai, P. Glackin, G. Silcock, and A. Abu-tair, "Structural Performance Of High Strength Concrete Columns In Fire," *Fire Safety Science*, vol. 7, pp. 1001–1012, 2003.

[53] T. T. Zhu, H. L.; Lie, "Fire Resistance Evaluation of Reinforced Concrete Structures," *Internal Report (National Research Council Canada. Institute for Research in Construction)*, pp. 45–49, 1993.

[54] V. Kodur and R. Mcgrath, "Fire Endurance of High Strength Concrete Columns ," pp. 73–87, 2003.

[55] M. Guenfoud and a. Benmarce, "Experimental behaviour of high-strength concrete columns in fire," *Magazine of Concrete Research*, vol. 57, pp. 283–287, jan 2005.

[56] M. Mohamed Bikhiet, N. F. El-Shafey, and H. M. El-Hashimy, "Behavior of reinforced concrete short columns exposed to fire," *Alexandria Engineering Journal*, vol. 53, pp. 643–653, sep 2014.

[57] W.-C. Jau and K.-L. Huang, "A study of reinforced concrete corner columns after fire," *Cement and Concrete Composites*, vol. 30, pp. 622–638, aug 2008.

[58] Y. y. Xu and B. Wu, "Fire resistance of reinforced concrete columns with L-, T-, and +-shaped cross-sections," *Fire Safety Journal*, vol. 44, no. 6, pp. 869–880, 2009.

[59] V. Kodur, D. Hibner, and A. Agrawal, "Residual response of reinforced concrete columns exposed to design fires," *Procedia Engineering*, vol. 210, pp. 574–581, 2017.

[60] K. H. Tan and Y. Yao, "Fire Resistance of Reinforced Concrete Columns Subjected to 1- , 2- , and 3-Face Heating," vol. 130, no. 11, pp. 1820–1828, 2005.

[61] A. Law and M. Gillie, "Interaction diagrams for ambient and heated concrete sections," *Engineering Structures*, vol. 32, no. 6, pp. 1641–1649, 2010.

[62] V. Kodur and N. Raut, "A simplified approach for predicting fire resistance of reinforced concrete columns under biaxial bending," *Engineering Structures*, vol. 41, pp. 428–443, 2012.

[63] S. F. El-Fitiany and M. A. Youssef, "Interaction diagrams for fire-exposed reinforced concrete sections," *Engineering Structures*, vol. 70, pp. 246–259, 2014.

[64] U. Bajc, M. Saje, I. Planinc, and S. Bratina, "Semi-analytical buckling analysis of reinforced concrete columns exposed to fire," *Fire Safety Journal*, vol. 71, pp. 110–122, 2015.

[65] P. Bamonte and F. Lo Monte, "Reinforced concrete columns exposed to standard fire: Comparison among different constitutive models for concrete at high temperature," *Fire Safety Journal*, vol. 71, pp. 310–323, 2015.

[66] V. Kodur, W. Khaliq, and N. Raut, "An approach to account for tie configuration in predicting fire resistance of reinforced concrete columns," *Engineering Structures*, vol. 56, pp. 1976–1985, 2013.

[67] D. V. Reddy, K. Sobhan, L. Liu, and J. D. Young, "Size effect on fire resistance of structural concrete," *Engineering Structures*, vol. 99, pp. 468–478, 2015.

[68] P. Kumar and V. K. Kodur, "Modeling the behavior of load bearing concrete walls under fire exposure," *Construction and Building Materials*, vol. 154, pp. 993–1003, 2017.

[69] P. Turgut and O. F. Kucuk, "Comparative Relationships of Direct, Indirect, and Semi-Direct Ultrasonic Pulse Velocity Measurements in Concrete," *Russian Journal of Nondestructive Testing*, vol. 42, no. 11, pp. 1061–8309, 2006.

[70] "Publication Is Licensed To," 2012.

[71] British Standards Institution, "Testing concrete ," vol. 3, 1992.

[72] a. Y. Nassif, E. Burley, and S. Ridgen, "A new quantitative method of assessing fire damage to concrete structures," *Magazine of Concrete Research*, vol. 47, no. 172, pp. 271–278, 1995.

[73] P. Cioni, P. Croce, and W. Salvatore, "Assessing fire damage to r.c. elements," *Fire Safety Journal*, vol. 36, no. 2, pp. 181–199, 2001.

[74] M. Colombo and R. Felicetti, "New NDT techniques for the assessment of fire-damaged concrete structures," *Fire Safety Journal*, vol. 42, no. 6-7, pp. 461–472, 2007.

[75] L. Biolzi, S. Cattaneo, and G. Rosati, "Evaluating residual properties of thermally damaged concrete," *Cement and Concrete Composites*, vol. 30, no. 10, pp. 907–916, 2008.

[76] H. Yang, Y. Lin, C. Hsiao, and J. Liu, "Evaluating residual compressive strength of concrete at elevated temperatures using ultrasonic pulse velocity," *Fire Safety Journal*, vol. 44, no. 1, pp. 121–130, 2009.

[77] Y. Lin, C. Hsiao, H. Yang, and Y. F. Lin, "The effect of post-fire-curing on strengthvelocity relationship for nondestructive assessment of fire-damaged concrete strength," *Fire Safety Journal*, vol. 46, no. 4, pp. 178–185, 2011.

[78] I. Hager, H. Carré, and K. Krzemie, "Damage Assessment of Concrete Subjected To High Temperature By Means of the Ultrasonic Pulse Velocity Upv Method," no. January, 2013.

[79] G. Trtnik, F. Kavčič, and G. Turk, "Prediction of concrete strength using ultrasonic pulse velocity and artificial neural networks," *Ultrasonics*, vol. 49, no. 1, pp. 53–60, 2009.

[80] S. Lai and M. Serra, "Concrete strength prediction by means of neural network," *Construction and Building Materials*, vol. 11, pp. 93–98, mar 1997.

[81] I.-C. Yeh, “Modeling of Strength of High-Performance Concrete Using Artificial Neural Networks,” *Journal of Civil Engineering*, vol. 28, no. 12, pp. 1797–1808, 1998.

[82] W. Dias and S. Pooliyadda, “Neural networks for predicting properties of concretes with admixtures,” *Construction and Building Materials*, vol. 15, pp. 371–379, oct 2001.

[83] S.-C. Lee, “Prediction of concrete strength using artificial neural networks,” *Engineering Structures*, vol. 25, pp. 849–857, jun 2003.

[84] L. Bal and F. Buyle-Bodin, “Artificial neural network for predicting drying shrinkage of concrete,” *Construction and Building Materials*, vol. 38, pp. 248–254, 2013.

[85] S. Chithra, S. R. Kumar, K. Chinnaraju, and F. Alfin Ashmita, “A comparative study on the compressive strength prediction models for High Performance Concrete containing nano silica and copper slag using regression analysis and Artificial Neural Networks,” *Construction and Building Materials*, vol. 114, pp. 528–535, 2016.

[86] Q. Zhou, F. Wang, and F. Zhu, “Estimation of compressive strength of hollow concrete masonry prisms using artificial neural networks and adaptive neuro-fuzzy inference systems,” *Construction and Building Materials*, vol. 125, pp. 417–426, 2016.

[87] A. Khashman and P. Akpinar, “Non-Destructive Prediction of Concrete Compressive Strength Using Neural Networks,” *Procedia Computer Science*, vol. 108, no. June, pp. 2358–2362, 2017.

[88] R. P. Pathak, B. K. Munzni, P. Sharma, N. V. Mahure, S. Vyas, and M. Ratnam, “Estimation of Fire Damage to Concrete Structure : A Case Study Field investigation,” vol. 2, no. April, pp. 130–136, 2013.

[89] S. Stanier, J. Blaber, W. Take, and D. White, "Improved image-based deformation measurement for geotechnical applications," *Canadian Geotechnical Journal*, vol. 53, no. 5, pp. 727–739, 2016.

[90] S. A. Stanier, J. Blaber, W. A. Take, and D. J. White, "Improved image-based deformation measurement for geotechnical applications," *Canadian Geotechnical Journal*, vol. 13, no. October 2015, pp. 1–35, 2015.

[91] L. Qixian and J. H. Bungey, "Using compression wave ultrasonic transducers to measure the velocity of surface waves and hence determine dynamic modulus of elasticity for concrete," *Construction and Building Materials*, vol. 10, no. 4, pp. 237–242, 1996.

[92] J. Ingham and F. Tarada, "Turning up the heat full service fire safety engineering for concrete structures," pp. 27–29, 2007.

[93] BS EN 1991-1-1:2002, "Eurocode 1 : Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings," *BRITISH STANDARDS INSTITUTION.*

[94] BS EN 1990:2002, "Eurocode 0 - Basis of structural design," *BRITISH STANDARDS INSTITUTION.*