Alternative Solutions for Canadian Fire Engineering Design

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Abstract

Fire Safety Engineering is important for resilient infrastructure design. Canadian structural fire design is currently restricted by its reliance upon prescriptive approaches. This research is the first step in the development of generalized frameworks from which practitioners can create Canadian alternative solutions. A methodology to develop an acceptance criterion for the fire design of unbonded post-tensioned concrete slabs is outlined. A stress relaxation model was used to establish preliminary definitions of critical design thermal boundaries. The analysis illustrated the need to consider travelling and localised design fires due to the vulnerability of unbonded tendons to localized heating. The fire performance of steel beam-to-column connections was then considered experimentally. This research program was the first step that will lead to the generation of analytical tools and further guidance regarding steel connections in fire. The results of the study provided preliminary guidance towards updating Annex K within CSA S16-19.

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Declaration

The work presented herein has been modified from existing publications written by the author and were influenced by input from several colleagues and co-authors. For the following original research article, which was modified and expanded into Chapter 3, the author conducted the analytical research and drafted the majority of the document. The co-authors provided to the development of the focus and conclusions of the study.

Chapter 3 is based on:

Jeanneret, C., Gales, J., Kotsovinos, P., and Rein, G. (2020) Acceptance Criteria for Unbonded Post-Tensioned Concrete Exposed to Travelling and Traditional Design Fires. *Fire Technology*. 56: 1229-1252.

The author has also written or co-authored additional publications which were partially included and/or referenced within this thesis. For each of these publications, the author was involved in the drafting of the documents and conducting the analytical research. These publications are included in the thesis appendices:

(A) Gales, J., Chorlton, B. and Jeanneret, C. (2021) The Historical Narrative of the Standard Time and Temperature Heating Curve for Structures. *Fire Technology*. 57: 529-558.

(D) Jeanneret, C., Nicoletta, B., Gales, J., Robertson, L., and Kotsovinos, P. (2021) Guidance for the post-fire structural assessment of prestressing steel. *Engineering Structures (Elsevier*).;

(G) Arces, S., Jeanneret, C., Gales, J., Antonellis, D., and Vaiciulyte, S. (2021) Human Behaviour in Informal Settlement Fires in Costa Rica. Safety Science (Elsevier) 142

Table of Contents

List of Figures

Chapter 2

Chapter 3

Chapter 4

List of Tables

Chapter 1:
Introduction

1.1 General

Fire Safety Engineering has become an important and prevalent field in recent years because of major events such as, but in no way limited to, the 2016 Fort McMurray Wildfire in Canada and the 2017 Grenfell Tower fire in London, UK. With architectural styles rapidly changing and the knowledge of fire behaviour constantly evolving, it is important to continue advancing the field of Fire Safety Engineering to ensure the existing and expanding infrastructure is safe and resilient. It has been observed that fires in large open spaces do not behave according to the assumptions used in the commonly used current design methodologies [3]. The current predominant method of fire design in Canada uses the CAN/ULC-S101-14 standard fire [4] in its prescriptive approach. This temperature-time curve assumes uniform burning and homogeneous temperature conditions throughout the compartment regardless of the compartment size, as well as other assumptions that are no longer valid as the standard was first codified in 1917 and is unrepresentative of contemporary fire dynamic theory [5]. Using such a curve can result in the building response not being entirely representative of reality, resulting in potentially unconservative or even overly conservative and wasteful designs.

Within the National Building Code of Canada (NBCC) [1] and subsequent provincial and material design standards, there exists flexibility for a designer to consider more advanced computational practices that can optimize the fire protection design. These clauses permit alternative design solutions to be used when they can be proven to be equivalent or superior to the prescriptive design. This, however, can be hard to implement regarding structural fire designs as the Authorities Having Jurisdiction (AHJs) typically do not have the knowledge and resources to evaluate and compare such a design [6]. Structural fire designs can be computationally intensive and training regarding fire design in Canada is limited. Only 5 universities in Canada offer courses regarding structural fire design, and these are mostly for graduate students [7].

Alternative solutions, especially for structural fire design, allow for economic and material savings. For example, a performance-based fire design of a steel structure might identify that some steel members do not require fire protection due to tensile membrane action (load carrying mechanism that occurs when a tension field and peripheral compression ring are created) or force redistribution when subjected to fire. It has also been identified that prescriptive design procedures can result in over or under designed structures, depending on the scenario. A performance-based solution is able to check for all possible scenarios and incorporate the fire protection needed in the specific areas identified. Some of the material design standards in Canada, such as CSA S16-19 Steel Design Handbook, incorporate some guidance for the practitioners to undertake an alternative solution, however this is currently limited and relies on the practitioners having an underlying understanding of structural fire behaviour and design [8].

With a focus on public safety, it is critical, through research, to expand the understanding of fire safety methods. While the Canadian field of fire engineering is growing, it is currently based on its American counterpart, limiting the development of new design methods. There is a need to expand the knowledge of the fire performance, which would help increase the confidence of engineers designing structures, as well as the Authorities Having Jurisdiction who approve the design. The concept of fire resilience is relatively novel in Canada, and there is a need to provide the required information to consultants to be able to achieve it.

1.2 Motivation

Alternative solutions in structural fire design have the ability to provide greater architectural freedom due to alternative protection and design solutions at a reduced cost. Using performance-based design can also help reduce the environmental impact of the design by optimizing the fire protection and reducing the need for excess fire protection, which can be toxic and have high cradle-to-grave impact. They also have the ability to demonstrate to a practitioner the life safety, economy and robustness benefits available with various fire solutions, as multiple different fire scenarios would be considered. This ensures that the designed structures are resilient.

Resilience is an important concept to address when it comes to structural design as a structure must be able to absorb, deform and recover, especially when there is a possibility that extreme events, such as fire, could occur. It is especially important for infrastructure where life-safety is concerned, as the structural systems are vital for structural integrity and can lead to collapse if improperly designed. It is therefore necessary for critical infrastructure to be resilient and be capable of resisting thermal loading. This has led to a push in the fire engineering community to optimize the design of structures by using performance-based fire-designed solutions instead of typical prescriptive design solutions.

The benefit to the sustainable design of structures is clear with the appropriate optimization and reduction of chemically laden fire protection materials. Performance-based design, however, involves the use of complicated software and cumbersome requirements to run the software. Hand calculations, such as the Bailey-BRE method [9] for steel structures, are not sufficient for advanced design techniques. Additionally, with many projects looking to Building Information Modelling (BIM) (which does not effectively communicate with existing fire design software [10]), the resulting fire design is usually in need of re-computation for each building iteration due to architectural changes. This can result in the fire design becoming more expensive than the actual material savings [11]. There is a need to develop computational tools that are both user friendly and have time saving abilities in calculations to optimize this fire protection.

There have been a few attempts at simplifying alternative solutions by developing acceptance criteria or hand calculations to simplify a complete fire design of certain systems. These, however, have been limited to select materials and structural systems, and are limited in their development description, making it harder for others attempting to develop acceptance criteria for different systems as they do not have a framework to start with. There is therefore a need to develop a generalized framework from which researchers, modellers and designers can use to create alternative solutions. This thesis investigates how alternative solutions for common structural systems can be developed, allowing for future designs to incorporate alternative solutions more easily. Overall, this will result in more resilient, safer and economical structural fire designs.

1.3 Scope of project

Fire can lead to structural damage in buildings constructed out of all types of materials. Performance-based design should be considered for all buildings to ensure resiliency through optimized solutions. There has been limited research published and disseminated regarding the development of alternative solutions, especially for a Canadian context. The basis of this thesis is therefore focused on exploring and developing alternative solutions that could be incorporated within national codes. The research presented in this thesis explores two interrelated themes:

(1) the behaviours of structural systems when subjected to fires that differ from the standard fire; and

(2) the structural and thermal response of connections when the structural system is exposed locally to high temperature.

To address the first theme, numerous analyses of a common unbonded post-tensioned steel (UPT) concrete floor plate system were performed for various different design fires to demonstrate the development of a modified acceptance criterion. Building on previous research that had identified potential failure modes for this type of floor system when exposed to localized fires [2], there was a need to identify the methods of failure for this system when exposed to various different types of fires. With the knowledge that different structural systems can have weaknesses when exposed to high temperature, this analysis investigates how different fires can affect a system. One of the common structural weaknesses in fire of a building are the connections. A preliminary series of fire tests was undertaken, with fires of different durations, to understand the deformation behaviour of a simple steel structure and how it dissipates the forces and heat into the connections. With an accurate understanding of the thermal forces created by a localized fire, design of connections would be able to correctly account for the load transfer. The tests will also demonstrate how connections and the remaining structure behave intrinsically when exposed to the thermal forces, such as displacements and rotations. This would allow for more accurate modelling of structures when exposed to fire.

These are critical steps for effective and resilient design of structures when exposed to thermal loading. It will allow for the proposal of safe design methods that consider the effects of the surrounding structure and realistic fire scenarios.

1.4 Research objectives

The current design methods for structures in fire in Canada are purely prescriptive, which results in designs often being either under-designed or over-designed. While there are options for performance-based design, this research project targets current significant knowledge gaps in the fire performance of several structural systems/materials. The results of this project will allow greater architectural freedom for structures that is not typically possible with the current prescriptive design. The lack of knowledge of how structural systems behave in fire has resulted in resistance to transfer to performance-based designs. This research, however, will help increase the confidence of engineers designing steel structures, as well as the Authorities Having Jurisdiction. The concept of fire resilience is relatively novel in Canada, and there is therefore a need to provide the required information to consultants to be able to achieve it. The research in this thesis thus aims to:

- study the available fire design methodologies in Canada and internationally, examining prescriptive and alternative solution designs to understand where limitations exist and where improvements are needed;
- develop an understanding of the range of fires that needs to be considered within a structural fire design, which exceed the typical assumption of a standard fire;
- perform preliminary fire tests on a beam to column connection setup to identify and begin to understand the structural actions that occur (i.e., thermal expansion, restraint etc.) within steel connections subjected to fire; and
- define how Canadian modellers and designers can attempt to incorporate alternative fire designs into their practice by generating preliminary guidance to undertake these performance-based designs.

1.5 Outline of thesis

This thesis has been written using portions of text that have previously been published in peer-reviewed journals but have been modified herein to maintain flow and coherence. The first drafts of these publications were principally written by the author during her Master's studies at York University. The following is a brief outline of each chapter:

Chapter 2: A Review of Structural Fire Design reviews the structural fire design methodology for various materials within a Canadian and international framework. The discussion is focused on the assumptions made in the prescriptive designs and how alternative solutions have recently been incorporated into Canadian codes and standards. Also discussed is the fire dynamics typically experienced within compartments, how this has changed over time, and how standards have changed in response. The intent of this chapter is to provide a general understanding of structural fire design in Canada for concrete and steel structures and identify urgent research gaps in the current methodologies. The latter is the focus of the work presented in subsequent chapters, with the aim of developing an acceptance criterion for design, and developing tools and technologies for data to reinforce the criterion.

Chapter 3: Defining Acceptance Criterion develops the current fire design methodology for unbonded post-tensioned concrete with emphasis on the acceptance criterion. This structure type is discussed as it is a commonly used concrete structural system, challenged by its vulnerability to local and travelling fire heating. There is a need to define a safe design criterion to prevent failure with such a system. This requires fire design methodologies to evolve concurrently to adapt to changing fire conditions. These methodologies allow for a range of fires to be considered within compartments, including fires that might not fully engulf the compartment. The effect of these various different design fires, including traditional design fires, on flat-plate unbonded post-tensioned (UPT) concrete, a structural system which allows long flooring spans (>8m), is analyzed and discussed. Two types of prestressing steel are analyzed for various unbonded length, and a real case study structure is used to demonstrate the development of a modified acceptance criterion. The focus of these analyses is to examine the unique deformation mechanisms present when prestressing steel is heated locally, an effect only observed when fires do not engulf the compartment. The range of fires considered allows for the demonstration of potential conditions that could be more onerous structurally that may be missed when only using traditional design. Additional experimental data examining prestressing steel post-fire is discussed, illustrating that traditional guidelines do not account for metallurgical changes due to the duration the steel was exposed to higher which can reduce the strength and cause failure.

Chapter 4: Connections in Fire focuses on technology generation for designing fire performance of steel beam-to-column connections. These connectors are known as the most vulnerable part of the steel framed building construction. Failure of steel beam-to-column connections within these structures under thermal loading is not well understood and technologies to model these responses are badly needed. Experimental data is necessary for the calibration and validation of computational modelling technologies. A series of three fire experiments, of various thermal exposure durations, is presented and discussed, aiming to produce preliminary validations of modelling. The focus within the experiments was identifying the thermal distribution into the connections when a steel beam was subjected to a localized pool fire at its center, as well as the deformations observed due to thermal expansion within the beam and connections. Recommendations for modifications to the current steel fire design guidelines in Canada are then developed.

Chapter 5: Conclusions and Recommendations outlines the main conclusions, novelty and relevance that were developed within the research presented, provides recommendations for modellers and designers attempting alternative designs within Canada, and summarizes areas that would benefit from future research to move the structural fire community in Canada towards developing and accepting alternative fire design solutions.

Chapter 2:
A Review of Structural Fire Engineering Design

2.1 General

The purpose of this chapter is to provide a general understanding of structural fire design in Canada and placing it into an international context. The current Canadian building code is an objective-based code, meaning that designs must meet the objective and functional statements which are tied to every prescriptive clause [1]. These objective and functional statements describe the goals of the code, without being quantitative. Alternative solutions are therefore allowed, which is where performance-based design is incorporated, however it is up to the practitioner to demonstrate "equivalency" such that the design performs at least as well as the acceptable solution given in the prescriptive clauses [12]. While these alternative solutions are options for structural fire designs, there are limited case studies published about structures in Canada that have been designed with performance-based designs [13]. These designs, while determining the performance of the structure through modeling, used failure temperatures (or limiting temperatures) determined using the standard fire and are therefore still relying on components of prescriptive methods. This is due to the requirements of showing "equivalency" to the prescriptive clauses within the code, causing the practitioners to limit the scope of their performance-based design.

The standard fire is a commonly and globally applied method to obtain the fire resistance metric. Its advantages lie in its simplicity, convenience in repeatability, and the fact that it has been used for more than a century with little to no change [5]. It therefore has a wealth of experience for testing performance of structures and building products. Contemporary architecture, however, demands structural engineering systems which can permit large open spaces (examples include airports, theatres, stadia, condos, offices, etc.). A number of real fire events have shown that fires in large open spaces will not result in uniform thermal exposure to the structure as is conventionally assumed [14]. In reality, fires can exhibit characteristics that demonstrate a "travelling" behaviour [15]. This has required an evolution within design methods to capture the realistic behaviour of fires and structures when exposed to the various types of possible fires. Ensuring the resiliency and safety of structure created the need for designs that account for structural performance and the many variables that affect it. In the transition towards performance-based design, it is becoming more important to model and understand structures exposed to a range of likely and expected design fires. This is typically done by selecting realistic design fires (based on engineering judgement, local practice or probabilistic studies) to ensure a robust structural design for fire.

This chapter will provide a high-level review of existing design fires and design methodologies considered within Canadian and international fire designs. The intent is to contextualize and demonstrate the changes needed to the currently heavy prescriptive design methods in Canada, while providing methods taken by jurisdictions currently using performance-based design. The objective-based code in Canada is showing the possibility of a shift towards performance-based design, allowing practitioners and Authorities Having Jurisdiction time to acquaint themselves with the processes required. The following review is intended to outline the changes previously made and identify areas that can be improved within following code revisions.

2.2 Prescriptive versus Performance-Based Design

2.2.1 The Standard fire and Prescriptive Design

The narrative of the origins of fire resistance and prescriptive design are best described within a recent paper in Fire Technology co-written by the thesis author. That narrative [5] is summarized herein, and can be found in Appendix A.

The contemporary definition of "fire resistance" was termed in the late 19th century. This came from an evolution of moving from defining building materials as fire-proof (restriction of combustible materials exposed to high temperature) to fire resistant (how any material performs in a fire – i.e., materials, even incombustible ones, will eventually fail and other metrics of analysis are needed beyond just stability – integrity and insulation for example). The terminology evolved from the aftermath of various city conflagrations. Examination of any major city illustrates these conflagrations, particularly those seen in Chicago in 1872, and Boston in 1874. The outcome of these city fires led to a surge in ad-hoc (ad-hoc meaning that they were demonstration in test design not necessarily following an established test methodology) fire tests of building elements that were often not trusted by the scientific and greater community.

The fire resistance design of structures is predominantly based on the adoption of the ASTM E119 standard fire test [16] that has its known origins more than 100 years ago. The adoption of the standard fire makes it possible to establish a qualification-based fire resistance metric for classification purposes, and the comparative performance of structures and other building components (doors, fire stops etc.).

The structural fire tests from which the standard time-temperature curve was developed were performed at the end of the 19th century to compare the fire resistance of different building materials that claimed to be "fireproof". This idealised curve, shown in Figure 2.1, was believed to be the worst-case fire an enclosure could be exposed to; however, it was created before fire dynamics were fully understood. By the late 1920s, it became known that the standard fire was not representative of reality. Its slow growth rate, its lack of a decay period and its autonomy from the compartment geometry, ventilation and fuel load did not agree with the behaviour observed during fire experiments. By the late 1970s and into the early 1980s, over-reliance on standard fire resistance testing was widely recognized as limiting innovation in architecture and construction, and technical papers began to appear which openly questioned the applicability of these tests. In the 1980s, Margaret Law questioned the standard fire curve's rationality and applicability in [17]. Her research, as well as those of multiple published researchers, showed that the fire dynamics in a compartment contradicted what was implied in the standard fire.

There exists no publicly available nor digitized documentation that explicitly defines the origins of the standard temperature-time heating curve that was created in 1916 and still used today to assess fire resistance. Within the current ASTM standard [16], a brief historical narrative is provided in its commentaries and the reader is directed to an original historical review of fire resistance testing that was presented in the Fire Technology journal over forty years ago [18], [19]. That two-part paper represents a historical narrative of standard fire resistance testing by Babrauskas and Williamson, titled: The Historical Basis of Fire Resistance Testing [18], [19]. These papers referenced in the standard examined the multitude of tests upon which it is assumed the standard temperature-time heating curve is based upon. Minimal if any changes to the standard time-temperature heating curve were made through the years in various iterations of ASTM standards that were produced after 1916. The test procedure itself showed increasingly less emphasis on residual capacity of the elements after a fire and to rather refine technological advances for test control (ensure more uniformity in heating for example). Overall, the fire community has largely followed the original testing procedure for construction materials and elements.

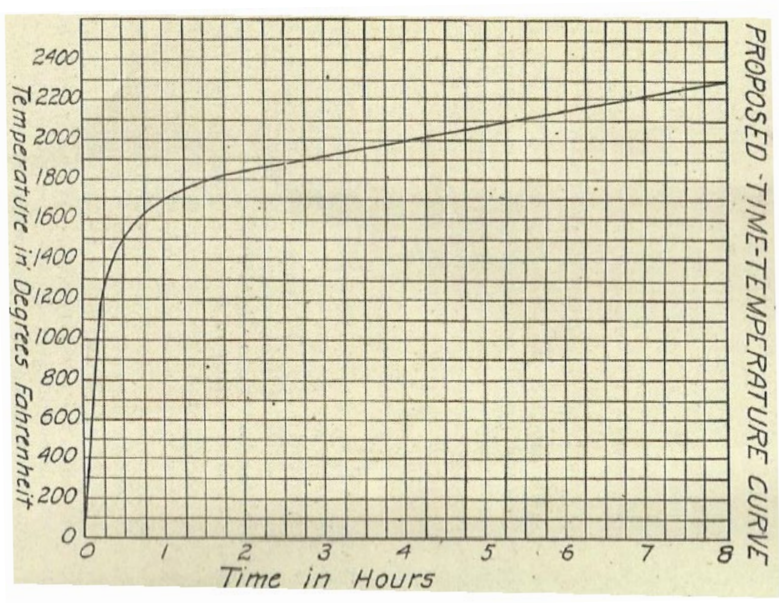

Figure 2. 1*:* Time-Temperature Curve Proposed in 1916 [20].

It is, however, still being used in many jurisdictions to determine the Fire Resistance Rating (FRR) of assemblies. The testing methodology, described in ASTM E119 or its Canadian equivalent CAN/ULC-S101-14, exposes the assembly to the standard temperature-time curve (shown in Figure 2.1) until it experiences certain failure criteria (i.e., stability, integrity…). The time before reaching failure is determined to be the assemblies' Fire Resistance Rating. While using these ratings, it is the responsibility of the practitioners to identify if the assembly under question should be considered "restrained" or "unrestrained". This has been identified by LaMalva *et al*. [21] as a source for unconservative design. As explained within [21], the definition of these "restrained" or "unrestrained" boundary conditions has yet to be provided by ASTM E119, leaving those using this testing method to interpret it to their best knowledge. The need for types of boundary conditions stems from the observation that the behaviour of structural members and assemblies during a standard fire test can be significantly influenced by the end restraint provided by the furnace frame. While the conditions have not been formally defined, an assembly is often considered "restrained" if it bears directly against the edges of the furnace at the beginning of the test and the thermal expansion of the assembly is limited. "Unrestrained" is therefore defined by the assembly being free to thermally expand without touching the furnace edges.

In furnace testing of an assembly, the components would likely be restrained equally at all boundaries, i.e., they would either be fully "restrained" at each boundary or allowed to thermally expand. In an actual structure, the system behaviour can differ and often the thermal restraints govern the response of the structural system in fire. The system can develop secondary load bearing or load redistribution mechanisms. For example, steel beams are often capable of developing catenary action if the connections are capable of resisting large tension forces, which increases the resistance of steel structures to collapse in fire. This type of behaviour cannot be realistically captured in furnace tests. These conditions are not representative of real structures however they have been incorporated into how practitioners can define the Fire Resistance Rating for the selected assembly, with higher ratings for restrained assembly. This allows for less fire protection to be applied if the practitioner determined the assembly would be “restrained” by the surrounding structure.

The standard fire may have its place for fire design purposes; however, it also has been recognised that it does not necessarily resemble the real fire behaviour that structures can experience. When considering the real fire behaviour expected in real structures, a range of responses should be expected depending particularly on the architectural geometry which differs from the compartments used to develop the standardized fire exposure.

2.2.2 Consistent Crudeness Principle

Real buildings and real fires are very complex. This complexity is often over simplified when performing standard fire tests. More importantly, a building’s complex behaviour in fire can be oversimplified in design if relative and consistent measures of complexity are not taken where fire and response to fire are considered. The fire safety in buildings is currently demonstrated using principles developed more than a century ago, including the standard fire.

The principle of ‘consistent crudeness’ was originally coined by Platt *et al.* [22] to describe a defensible approach to computational modelling of structural response to fire. Since the thermal exposure assumed in the structure can be crude in its actual representation of a fire, the modeler should use a similar degree of crudeness in performing the structural analysis for fire. For example, using a standard fire thermal exposure on a detailed multi-storey finite element model of a building lacks a consistency of crudeness [14]. This same principle should be applied when developing and performing large-scale non-standard fire tests [23].

Figure 2.2, recreated and updated from [14], schematically demonstrates how the principle of ‘consistent crudeness’ could be applied to structural fire testing. The vertical axis of the matrix lists the different complexities available for the thermal exposure. This ranges from a simple, steady-state fire to transient heating all the way to a real fire. The horizontal axis lists the different complexities of structural test assemblies or configurations. This ranges from testing the material to testing a real, three-dimensional structure.

From this matrix, consistent crudeness is shown along the diagonal arrow. This ensure that the complexity in both the structural configuration and thermal exposure remains relative. The arrow leads to the 8:5 entry where a real building is exposed to a real fire, the most complex and accurate testing method. Bisby *et al*. [14] also included in the matrix where large-scale fire tests that have been performed in the past few decades would fit. The table including all the fire test has been recreated below, and further information regarding the matrix and experiments can be found in [14].

Bisby *et al*. noted while creating this matrix that only a few structural engineers understand fire science enough to know the importance differences between the fire models compared to most engineers understanding importance of testing a full assembly for its structural response [14]. This resulted in most large-scale structural fire tests over the years aimed to reproduce the standard fire instead of a real fire. In contrast, it was Bisby *et al.*'s belief that more understanding of structural behaviour would stem from simulating more accurate fires than increasing the complexity of the testing assemblies [14]. It should also be noted that, since the majority of the large-scale testing shown in Figure 2.2 occurs in the upper left corner of the matrix, where elements or assemblies are tested within a furnace, the real restraint and connection of the assemblies are seldomly captured.

Structural Model / Fire Model		Materials & Partial Elements	Single Elements	Sub-Frame Assemblies	Transiently Simulated Restrained Assemblies	Real Structures
Elevated Temperature Exposures (transient or steady-state)	Steady-state Transient-state	Generate design/model input data	**5 - 56**	**5 - 44**		**9a - 9b**
Standard Fires	ISO 834	Generate design input data	**24 - 25 - 27 28 - 47 - 48** Standard fire resistance tests	**1 - 3 - 10 19 - 22 -23 29 - 30 - 41 45**	**37**	**2 - 14**
Equivalent Fire Severity to a Standard Fire	ISO 834	Validation of fire severity concept	Fire resistance ratings (using alternative severety metrics)	**35**		**4 - 12**
Parametrically Defined Model Fires	Fire = f()	Generate design input data	**26**	**20 - 42 - 40 50**		**9c - 9d - 9e 15**
Localised Fires		Generate design input data	**49**	**39**		**13 - 46**
Zone Model Defined		Research		**36**		
Field Model Defined		Research				
Real Fires	Real fire	**23** Research	**31 - 32**	**6 - 7 - 8 - 11 17 - 21 - 33 34 - 38 - 51**		**9f - 16 - 18 43 - 52** REAL behaviour in a REAL fire

Figure 2. 2: 'Crudeness framework' which outlines the level complexity of both thermal exposure and testing frame. Recreated and updated with permission of authors [14].

Table 2. 1: Large-scale non-standard structural fire tests. Recreated and updated with permission of authors [14].

Test	Year	Name of test and/or research institute	Reference
1	1959	Unbonded post-tensioned and slab assembly, USA	Troxell, 1959 [24]
2	1982	AISI and NIBS, USA	Jeanes, 1982 [25]
3	1983	Multi-span continuous unbonded post-tensioned slabs, Belgium	Van Herberghen and Van Damme, 1983 [26]
4	1985	Stuttgart-Waihingen University Germany	British Steel, 1999 [27]
5	1986	Steel beams and portal frames with uniform heating, Germany	Rubert and Schaumann, 1986 [28]
6	1987	Steel portal frame with fire load of wooden cribs, BRE, UK	Cooke and Latham, 1987 [29]
7	1992	BHP-William Street, Australia	British Steel, 1999 [27]
8	1994	BHP-Collins Street, Australia	British Steel, 1999 [27]
9a	1996	BRE Cardington Steel Building (Test 1), UK	British Steel, 1999 [27]
9b	1996	BRE Cardington Steel Building (Test 2), UK	British Steel, 1999 [27]
9c	1996	BRE Cardington Steel Building (Test 3), UK	British Steel, 1999 [27]
9d	1996	BRE Cardington Steel Building (Test 4), UK	British Steel, 1999 [27]
9e	1996	BRE Cardington Steel Building (Test 5), UK	British Steel, 1999 [27]
9f	1996	BRE Cardington Steel Building (Test 6), UK	British Steel, 1999 [27]
10	1997	Punching shear test in standard testing furnaces Germany	Kordina, 1997 [30]
11	1998	Car Park Fire Tests, CTICM, France	Vassart and Zhao, 2011[31]
12	1999	BRE Cardington Timber Frame, UK	Lennon et al., 2000 [32]
13	1999	Tests on steel portal frame with pool fires, UK	Wong et al., 1999 [33]
14	1999	Tests on steel portal frames with furnace exposure, China	Zhao and Shen, 1999 [34]
15	2001	BRE Cardington Concrete Building, UK	Bailey, 2002 [35]
16	2003	BRE Cardington Steel Building (Test 7), UK	Wald et al., 2006 [36]
17	2006	Ostrava Fire Test, Czech Technical University, Czech Republic	Chlouba et al., 2009 [37]
18	2006	Dalmarnock Fire Tests, UK	Rein et al., 2011 [38]
19	2007	Harbin Institute of Technology, China	Dong and Prasad, 2009 [39]
20	2007	BRE Hollow-Core Slab Fire Test, UK	Bailey and Lennon, 2008 [40]
21	2008	Mokrsko Fire Test, Czech Technical University, Czech Republic	Chlouba and Wald, 2009 [41]
22	2008	FRACOF Fire Test, Metz, France	Vassart and Zhao, 2011 [31]
23	2008	CROSSFIRE Fire Test, Metz, France	Vassart and Zhao, 2011 [31]
24	2008	Long span post-tensioned slab tested in a large furnace, UK	Kelly and Purkiss, 2008 [42]
25	2008	Model continuous post-tensioned concrete slab standard fire, China	Li-Tang et al., 2008 [43]
26	2008	Steel frames tested under ‘natural’ fires, Portugal	Santiago et al., 2008 [44]
27	2009	Two-dimensional steel frames in custom furnace, China	Dong and Prasad, 2009 [45]

28	2010	Continuous post-tensioned concrete slabs in furnace, China	Zheng et al., 2010 [46]
29	2010	Planar portal frames in standard fire exposures, China	Han et al., 2010 [47]
30	2010	Hong Kong Fire Test, Hong Kong Polytechnic University, China	Wong and Ng, 2011 [48]
31	2010	CCAA-CESARE Test, Australia	CCAA, 2010 [49]
32	2010	University of Ulster, UK	Nadjal et al., 2011 [50]
33	2011	Punching shear tests in standard testing furnaces, Belgium	Annerel et al., 2011 [51]
34	2011	TU Munich Fire Tests, Germany	Stadler et al., 2011 [52]
35	2011	TU Vienna Fire Tests, Austria	Ring et al., 2011 [53]
36	2011	University of Edinburgh and IIE Roorkie Test, India	Sharma et al., 2012 [54]
37	2011	NRC, Ottawa, Canada	Mostafaei, 2011 [55]
38	2011	Veseli Fire Test, Czech Technical University, Czech Republic	Wald et al., 201 [56]
39	2013	Continuous post-tensioned concrete slabs, Edinburgh	Gales et al., 2016 [57]
40	2014	Precast CFRP Pretensioned HPSCC slabs	Maluk et al., 2014 [58]
41	2015	Performance concrete thin plates under ISO 834 fires	Hulin, T. et al., 2015 [59]
42	2016	Shear capacity of steel-sheathed cold-formed steel framed shear walls – Phase 1, NIST	Hoehler and Andres, 2018 [60]
43	2017	Obora X-ONE travelling fire test, Poland	Rauckauskaite et al., 2018 [61]
44	2018	Shear test of deep PCHC slabs under linear heating curve, Singapore	Nguyen and Tan, 2018 [62]
45	2018	Two way precast concrete under ISO 834, China	Xu et al., 2018 [63]
46	2018	Composite bridge testing, Universitat Politecnica de Valencia, Spain	Alos-Moya et al., 2018 [64]
47	2018	Concrete filled hollow columns under ISO 834, Portugal	Laim et al., 2018 [65]
48	2018	Unprotected composite beams pinned with steel girders under ISO 834, Japan	Dwiputra et al., 2018 [66]
49	2018	Long-span composite beams with gravity connections, NIST	Choe et al., 2018 [67]
50	2018	Shear capacity of steel-sheathed cold-formed steel framed shear walls – Phase 2, ASTM E119 and parametric fires, NIST	Hoehler and Andres, 2018 [60]
51	2018	High strength steel endplate connection under linear loading, Netherland	Qiang et al., 2018 [68]
52	2018	Steel portal frame structures using wood crib fires, China	Jiang et al., 2018 [69]

2.2.3 Contemporary Design Fires and Performance-Based Design

The current form of the standard fire curve reflects very accurately the time period during which the standard time-temperature heating curve was being developed (1890-1916). To suggest, however, that the standard temperature-time heating curve still serves its original purpose to this day is to argue that no advancements have been made in fire science, instrumentation, or even structural engineering since 1916 [5]. We, as an industry, can respect its origins and at the same time, we need to use our contemporary knowledge to build upon it to create the next iterations of standards. Methodologies capturing the possible behaviour of fires within large compartments are greatly needed in Fire Safety Engineering.

At this time, design methods in North America mostly still rely on the Fire Resistance Rating (FRR) determined using the ASTM E119 test method which incorporates the standard fire curve. In Europe, additional design curves have been accepted into practice such as the Parametric curves [70], and recently, there has been some use and acceptance of the Travelling Fire Methodology [71]. The Eurocode Parametric fire curves are based upon small residential-scale compartment tests. They assume that there is uniform burning and homogeneous temperature conditions within the compartment, regardless of the compartment's size. These assumptions mirror the standard fire curve, however the parametric curves also allow for the consideration of the compartment geometry, ventilation, and fuel load, as well as including a cooling phase. The parametric fire curves, however, are limited to floor areas less than 500 m^2, compartment heights less than 4 m, and lining materials with thermal inertias between 1000 and 2200 $J/m^2\ s^{1/2}\ K$, and there cannot be openings in the ceiling [70]. These limitations signify that the parametric curves cannot be used for spaces such as large compartments, atriums, and enclosures lined with modern materials such as glass and highly insulating materials.

Figure 2.3 details the evolution of travelling fire models, with a focus on the methodology used herein, iTFM. The origins of travelling fires stem from the observation of fires in large, open-plan enclosures, where the fire was seen to travel within the available space and last for extended durations [72]. One of the first approaches at simulating a travelling temperature-time curve was developed by Clifton [73] in New Zealand. This methodology divided each compartment, or fire cell, into four distinct regions (preheat, fire, burned out, and smoke logged) that were subjected to parametric fire curves individually and sequentially. The Travelling Fires Methodology (TFM), as known today, was introduced and developed in 2006 by Stern-Gottfried and Rein [3], [74] to capture the highly non-uniform and transient heating over the entire area of large compartments, building on the research of Rein *et al.* in 2007 [75]. The TFM methodology was developed further to become iTFM in 2015 by Rackauskaite *et al.* [71] to account for a flapping angle which captures the possible fluctuations of the impinging flame. Dai *et al*. [76] developed an Extended Travelling Fire Methodology (ETFM) that combined Hasemi's localized fire model with a calculation for the smoke layer. This method accounts for the accumulation of hot gases within the compartment. Travelling fires can lead to more severe thermal conditions and structural response than a uniform compartment fire, as shown by [77]. The development of these methodologies has led to the incorporation of travelling fires into the design process, shown through the UK standards PD7974-1 and -3 which now reference travelling fires as representative of industry practice [78], [79]. A detailed description of the calculations for the Parametric curves and the Travelling Fire Methodology can be found in Appendices B and C, respectively.

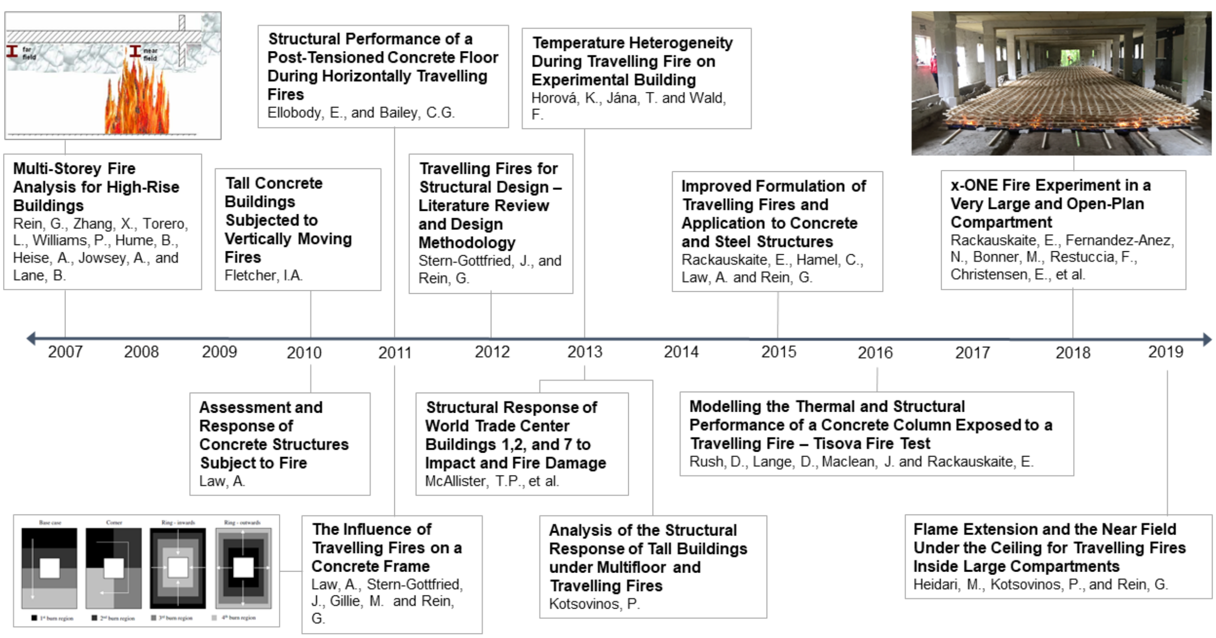

Figure 2. 3: Brief contextual contemporary history of the Travelling Fires Methodology in literature, as related to this study (post 2007).

The iTFM methodology was chosen as the representative travelling fire design fire in this thesis as it was developed for design purposes and is based on conservative assumptions. It is currently the most applied methodology for large compartments used in industry, with many new iconic buildings using it as part of their design. It is purposefully not aiming to be predictive due to the limited experimental evidence from large compartments. A very limited number of experiments have been undertaken in large enclosures and therefore, fire dynamics in such spaces require further research and a number of experimental campaigns are currently pursued by researchers internationally. It should be noted that, at this point, the iTFM design methodology, or any other design fire methodology for large enclosures, are not yet validated. It is acknowledged that other travelling fire methodologies could be considered.

All the time-temperature curves are shown in Figure 2.4 with comparison to the standard time-temperature curve. Two parametric curves were created, a short hot fire and a long cool fire that show the possible brackets of the curve. These are identified by changing the possible ventilation area, changing the fire from fuel to ventilation controlled. A representative travelling fire was also created for comparison. Travelling fires are usually considered as a family of fires, as the various sizes of the moving fire have to be considered (different floor area percentage covered by the flame). For this figure, a 20% iTFM was considered, which indicates that only 20% of the floor would be on fire at one time as the fire moves from one side of the room to the other. For the curve generation, a theoretical compartment of 20 m by 20 m, with a height of 3.6 m, was considered. A typical office fuel load was considered (511 MJ/m^2). The walls were assumed to be covered with gypsum, while the floor and ceiling were considered to be concrete. For the

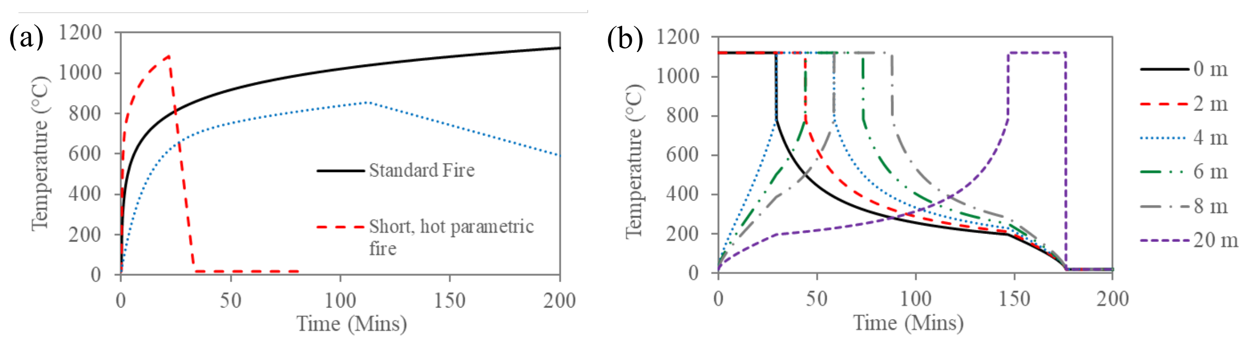

Figure 2. 4: Different design fire examples including (a) the standard fire, two sample parametric fires and (b) a sample travelling fire as it progresses through the room.

parametric fires, the short hot fire was assumed to have a ventilation area of 10 m^2, while the long cool fire was assumed to have a ventilation area of 160 m^2. For the travelling fire, the heat release rate per unit area was determined to be 290 kW/m^2. The temperature in the near field (within the fire area) was assumed to be 1200°C, and the flame was assumed to have a flapping angle of 6.5°. The assumptions for the travelling fire are typical of this type of analysis, as shown in [71].

From Figure 2.4, the difference between all of these curves can be clearly seen. The clear differences between the standard fire and the parametric fires are the variance possible for different ventilation and the consideration of the cooling phase. The overall shape of the heating curve between the standard fire and the parametric curves could be considered similar (ramping up over time) however parametric curves change for more rapid or slower heating depending on the available oxygen in the room. An important consideration of the parametric curves is the cooling phase, which also differs depending on the ventilation, and allow for more realistic consideration. This design fire also takes into account the type of materials the enclosure is composed of. When comparing the standard fire to the travelling fire, the shape and process of it is completely different. Within Figure 2.4b, the different curves illustrate how different locations within the compartment experience the fire temperatures. The curves from the left to the right (black, red, … to purple) follow the path of the fire, with the black curve being at the location of ignition (considered to be instantaneous) and the purple curve being at the location where the fire ends. The travelling fire shows different features to heating, such a preheating and gradual cooling, all while the fire is still occurring at a different location within the compartment.

These different curves are used for design internationally. Although they also have their limitations, they allow for more consideration of fire dynamics, as well as the fuel and oxygen available to the fire. While these curves could be used in North America as part of alternative solutions, there has been limited use [80]. Instead, alternative solutions for fire design have been mostly restricted to computational modelling, such as finite element analysis (FEA) and computational fluid dynamics (CFD). The limited use is due to the designs having been approved by the Authorities Having Jurisdiction (AHJ), who are unfamiliar with the new design curves. The parametric and travelling curves were developed as an acknowledgement that the standard fire no longer represented the current understanding of fires, and in some way, should no longer be used as a design fire.

2.3 Building Codes

As previously stated, the National Building Code of Canada (NBCC) is an objective-based code [1]. This type of code is a way to retain the prescriptive aspects of codes, called acceptable solutions, while allowing performance-based designs, called alternative solutions, if equivalency can be shown. A brief summary of the Canadian development on the objective-based code in context to its international counterparts is provided below. A detailed review of the evolution of the National Building Code of Canada with a focus on fire design requirements was undertaken by Hadjisophocleous *et al*. [81] for the evolution up to 1998 and by Smith [13] for the evolution to date. Details regarding structural fire design in Canada are then provided, in comparison to those used in Europe.

2.3.1 Objective-based Codes vs Performance-based Codes

Canadian involvement in fire research occurred from the beginning, with the Canadian Society of Civil Engineers (CSCE) and Underwriters Laboratories (UL) being involved with the creation of the standard fire in 1916. It was however not until after the 1940s that Canada started its own research into fire safety. Once it had established the National Research Council (NRC) in 1947, it identified fire as a required research topic to improve the construction industry [82]. The focus was initially on defining fire resistance ratings, using the standard fire, for various materials and assemblies to be provided within the National Building Code Supplement No. 2. In 1958, the NRC had the opportunity to perform large scale fire tests, in collaboration with the British Joint Fire Research Organisation (BJFRO) due to the need to demolish the buildings [15]. These fire tests were the basis for the requirements for spatial separation between buildings that was incorporated into the following iterations of the NBCC [82]. Around the 1980s, when NRC obtained a new facility that allowed for larger scale testing, the research expanded to the response of an entire building to fire instead of only single elements [82]–[84]. This coincides with the increasing use of performance-based design around the world as it was starting to be recognized that prescriptive codes were overly complex and did not capture the behaviour of the full structure. The focus was still however on the prescriptive side of fire safety in Canada.

The United Kingdom was the first country to adopt a performance-based code in 1985, switching its prescriptive clauses for functional statements that identified safe goals for structures [85]. Work at the NRC facilities in 1987 began the foundation for risk-based design methods by developing probabilistic modelling of fire risk, addressing human, fire and smoke behaviour [86]. This led to the performance-based codes, the National Building Fire Safety Systems Code, being adopted in 1991 in Australia [87]. These first versions of performance-based codes gave the practitioners lots of freedom to achieve cost effective fire protection, design flexibility, and quantified risk [88]. This however led to many challenges, especially for the Authorities Having

Jurisdictions in determining the quantified risk in designs that had no defined performance criteria. There was a lack of information supporting the novel design method for the authorities, requiring them to trust that peer reviews by other practitioners would ensure safety [89]. There was even a need for more education on the side of the practitioners, who were mostly trained on the prescriptive design method. To ease some difficulties due to the freedom given by performance-based design codes, New Zealand decided to implement guidelines that prescribe exact performance criteria and specify which design fire scenarios must be considered as a baseline [90]. This would help limit the varying risk between designs as practitioners were free to use differing assumptions and design fires within their designs.

Canada outlined its shift towards a more performance-based code in 1994, after being able to assess what had been done in other countries. It took this opportunity to learn and improve, resulting in the current objective-based code, which retains the prescriptive clauses as acceptable solutions and allows for innovation through alternative solutions. This objective-based code took ten years to complete and was implemented in 2005. Regarding fire safety, the current predominant method of fire design in Canada uses the CAN/ULC-S101-14 standard fire [4] in its prescriptive approach, even though alternative solutions are allowed. There is however a push towards the performance-based solutions, which are now commonplace in other countries, as there is growing concern that the prescriptive method is not always conservative as it does not consider the structural behaviour of the entire structure.

2.3.2 Canadian Fire Design

As previously described, the prescriptive method using Fire Resistance Ratings is currently the predominant method for structural fire design in Canada. Common assemblies and materials have been tested following the CAN/ULC-S101 methodology, and some have been listed within the NBCC for simple access. Within the NBCC, there is a factored load case that accounts for thermal effects within structure [1]:

$$D + Ts + (\alpha L \text{ or } 0.25S) \qquad \textit{(Eq. 2.1)}$$

where

D = dead load

Ts = expansion, contraction, or deflection effects due to temperature

α = 1.0 for storage, equipment, or services areas. 0.5 otherwise

L = occupancy live load

S = variable load due to snow

This case, however, is included in the commentaries of the code, not the main code. Using this equation requires advanced structural fire engineering knowledge and would be considered an alternative solution. The commentary does not define what design fire should be considered within the design, stating only that it must be "appropriate" and consider the forces generated by the structure when exposed to fire. A deterministic approach is currently used to determine which design fire to use. This includes a set series of design fire for which the performance of the structure is determine and compared against the required performance criteria. Fires are categorized as either localized, external or post-flashover (compartment) fires.

Provincial and territorial codes follow the guidance for fire design from the NBCC. More guidance, however, can be found in some of the material design standards, where calculation methods to achieve a fire resistance rating are outlined. The CSA S16-19 Design of Steel Structures has the most comprehensive details regarding structural fire design. The mandatory Annex K: *Structural design for fire conditions* outlines analytical methods, similar to those for ambient conditions, that can be used to check the capacity for individual steel members at elevated temperatures. It utilizes reduction factors to account for degrading material properties as the temperatures increase. There are calculations methods for tension, compression and flexural members. The factored case load can also be found within this annex. Additionally to the load combination method, the annex provides preliminary guidance on material properties and considerations that can be used in a performance-based design. It should be noted that the analytical calculations and properties outlined within Annex K are generally replicated from the Eurocodes, which are far more comprehensive. In fact, the Canadian code often refers the practitioner to international literature for calculations regarding fire design.

The concrete design standard, CSA A23.3-14 Design of Concrete Structures, does not contain instructions on fire design. Instead, it defers the practitioner to the applicable building code. The handbook in which the standard is published, however, does include a section that outlines the NBCC requirements and how to calculate the fire resistance ratings for different structural components. It is missing, however, any mention of analytical methods to calculate the capacity of structural members or how the concrete degrades with temperature. It should be noted that these degradation factors can be found within the CSA S16-19 Design of Steel Structures due to the commonly used composite construction, which are also adapted from the Eurocode.

The timber design standard, CSA O86-19 Engineering Design in Wood, does contain a section pertaining to fire design within Annex B: *Fire resistance of large cross-section wood elements*. The method used to ensure fire resistance includes increasing the dimensions on the timber so that the resistance is calculated using the reduced cross-section that occurs as wood chars.

2.3.3 European Fire Design

The countries within Europe adopted a generalized building code which allows for consistent construction across countries. This building code is performance-based, including the structural fire design. There has been a large amount of international case studies published, allowing for more insight and a more mature use of performance-based design. Selecting design fires is often done probabilistically, aiming to reach an acceptable level of risk instead of meeting set performance criteria. When deterministic approaches are used, a broad range of design fires are used to ensure various scenarios are covered. As mentioned in the previous sections, parametric and travelling fires are commonly used and it is specified in the design codes to account for realistic fire conditions such as a cooling phase. Behaviours such as contraction during cooling and the formation of tensile membrane action in composite steel structural, which have been demonstrated experimentally [27] and within accidental fires [91], are used within design. These provide many benefits, both architecturally and economically, as elements can remain exposed.

2.4 Summary

This chapter provided a review of structural fire engineering for both Canadian practice and internationally. Within the National Building Code of Canada (NBCC) [1] and subsequent provincial and material design codes, there exists flexibility for a designer to consider more advanced computational practices that can optimize the fire protection design. These clauses permit alternative design solutions to be used when they can be proven to be equivalent or superior to the prescriptive design. This, however, can hard to implement regarding structural fire designs as the Authorities Having Jurisdiction (AHJs) typically do not have the knowledge and resources to evaluate and compare such a design [6]. Some of the material design codes in Canada, such as CSA S16-19 Steel Design Handbook, incorporate some guidance for the practitioners to undertake an alternative solution, however this is currently limited and relies on the practitioners having an underlying understanding of structural fire behaviour and design [8]. The following chapter will begin to develop an acceptance criterion which will help practitioners undertake performance-based design.

Chapter 3: Defining Acceptance Criterion

3.1 General

Chapter 2 demonstrated the different fire design methods used in Canada, as well as internationally. This illustrated that designing using the standard fire, either as a design fire or by using Fire Resistance Ratings, cannot always capture the full possible extent of potential fires on a structure. Research studies have shown that “short, hot” and “long, cool” uniform parametric fires can lead to different responses [92] which cannot be observed in standard fire tests. As well, travelling fires can introduce a broad range of responses, which are thermally more severe, and the structural severity depends on the specific heating scenario [71], [77], [93]. To date, the real structures that have been designed using travelling fires methodologies are open plan offices of largely steel composite or traditional reinforced concrete configuration – mostly because the structural fire model validation exists for these structure types, i.e. the Cardington Studies of 1995 and 2003 [92]. However, other structural systems which also enable large compartmentation have seen limited application of the travelling fires methodology in practice.

Acceptance of a structural fire engineering design is subject to approval by the building authority, which may require a third-party review. Analytical tools are simplified tools for practitioners that allow for quick simple but conservative analysis to be conducted. An analytical tool would allow for a more conservative and certain design until further research and complex design guidance can be produced that improve our understanding of these structures in fire conditions. Analytical tools can become the basis of acceptance criteria, which should reflect the intent of the minimum performance objectives as stipulated in applicable regional standards (see [94]), as well as those discretionary performance objectives that are enacted for a given project (if any). While the United States has adopted performance objectives as specified ASCE/SEI 7-16 Appendix E, Canada has a lack of information available to practitioners and jurisdictional authorities for concrete structures [94]. Acceptance criteria should be quantitative measures to fulfill the required performance objectives, and should meet the minimum criteria that is defined, based on the structural system material type. The criteria are normally applied to the result of an analytical or numerical computed structural fire design.

Herein, the development of an acceptance criterion for a complex structural system, unbonded post-tensioned concrete is considered. Post-tensioned (PT) concrete uses highly stressed prestressing steel tendons to achieve long spans. Prestressing steel may take the form of strands or cables fabricated through using a series of wires that are high strength and cold drawn. In concrete, prestressing steel tendons are tensioned before (pre-tensioned) or after (post-tensioned) the concrete casting and released after the concrete has cured, creating an external compressive force on the concrete member. This process takes advantage of concrete's compressive strength and allows the structural member to sustain higher loads, achieve longer spans, and meet strict serviceability deflection limits. This also helps meet sustainability objectives by reducing the required concrete for construction.

Prestressing steel is highly sensitive to temperature changes which can invoke strength failure and other deformation characteristics. Some of the more prevalent concerns include the complex and rapid failure of the prestressing steel during fire when there is a sudden strand thermal exposure if the concrete cover spalls [95]. Subsequently, post-fire investigations of critical infrastructure must be conducted as soon as possible after a fire to assess structural integrity and limit severe economic losses. The difficulty with post-fire investigations in prestressed structures is the complex behaviour of the materials involved. Exposing prestressing steel to elevated temperatures can severely reduce mechanical properties and cause complex material changes related to its fabrication techniques. In addition, prestressing steel will not regain its mechanical properties as hot rolled steel does. As a result, a potential fire can have significant consequences in terms of post fire repair even if the event did not result in any collapse. Since the industry rapidly changes and refines these fabrication processes, prestressing steel is becoming more complex as a material, and understanding its behaviour in high temperatures is becoming increasingly difficult. There is very limited guidance to assess the fire damage in prestressing structures. There is even less guidance for non-destructive assessment of structures containing it as this requires understanding the post-fire condition of the prestressing steel.

Post-tensioned concrete structures under design fires which incorporate localised heating have received limited research attention. For post-tensioned concrete (PT) structures, Gales *et al.* [57] have shown experimentally that a localised fire can be more onerous in comparison to a uniform fire (an assumption of the standard fire test) due to the stiffer restraint of the prestressing steel from the unheated areas. The application of iTFM could be critical when considering that PT concrete has specific vulnerabilities to localised heating that can cause its high strength steel reinforcing to rupture – specifically if the steel is left unbonded to the surrounding concrete rather than bonded as discussed in a previous literature review on real fire case studies and experimental evidence [57]. Additionally, corresponding reductions in stress relaxation and tendon failure hampers the structures' ability to balance applied loading over long spans. By including travelling fires within the family of design fires and identifying the most critical heating scenarios, it becomes possible to design PT concrete structures so that failure mechanisms through tendon rupture or stress relaxation are minimized.

This chapter aims to develop the capabilities in modelling PT concrete for prestressing steel tendon relaxation and rupture by proposing a limitation on tendon length in design, rationally identifying the critical design fires necessary to ensure this type of structure's fire resilience, producing an analytical tool and concluding with a priority listing of research needs for PT concrete structures for industry and researchers moving forward. The intent is to provide guidance to practitioners in Canada for composite (reinforced concrete) as current CSA standards for fire design do not cover concrete or hybrid structures.

3.2 Literature Review of Post-Tensioned Flooring Systems in Fire

Unlike PT concrete structures, the behaviour of reinforced concrete structures at elevated temperatures has been extensively studied in previous literature. Khoury [96] provides an overview of the concrete material response at elevated temperatures as well as of the methods for achieving fire resistance for concrete structures. Dotreppe *et al.* [97] present an experimental study on the fire behaviour of concrete columns, and Lim [98] presents experimental and numerical studies of two-way slabs. A numerical model for the fire resistance of reinforced concrete beams is presented by Kodur and Dwaikat [99]. A multiaxial constitutive model for concrete at elevated materials has been developed by Gernay *et al.*[100], and Molkens *et al.*[101] have examined numerically the compressive membrane action developed in concrete slabs. However, research on PT concrete exposed to fire, and more specifically unbonded PT, has received little attention and therefore has developed a degree of documented uncertainty towards its performance in fire. A series of fire test for one-way spanning unbonded PT concrete were performed by Bailey and Ellobody [102] with the aim to understand its fundamental behaviour.

One of the most comprehensive structural fire experiment programmes that permits the study towards rational design of PT concrete in fire is the set of experiments performed by Gales *et al.* between 2011 and 2013 in the United Kingdom (see [57]). These experiments included testing of three multiple bay, realistically-restrained concrete slabs heated locally with radiant heaters that permitted a highly controlled and repeatable heat exposure between experiments. Two experiments were unbonded (noted as A and C) and one was bonded (noted as B). An unbonded tendon is free to deform along the length of the installation in the slab and is easier to construct, whereas bonded is more difficult to construct and can only deform locally. To study the effect of Load Induced Thermal Strains (LITS), two of these slabs were heated twice (A and B) while the third slab (C) was only heated once [103]. LITS occurs in concrete structures when they experience high temperatures for the first time and can cause an upward deflection in concrete slabs which would otherwise seem counterintuitive given that the deflection is contrary to the deflection of the slab due to the applied loads. Including a secondary heating has been shown to result in no additional LITS strain effects observed, provided that the stress and heating levels of the first heating are not exceeded. The experimental specimens were exposed to a long-cool fire under expected service load conditions. The first heating cycle included heating the slab until the prestressing steel reached approximately 350°C, a commonly used critical temperature in design standards, using a constant radiant heat flux of about 35 kW/m^2. For the slabs receiving a second heating cycle, the slabs were heated using the same heat flux until the prestressing steel reached 427°C (the temperature at which prestressing steel is considered to lose half of its in-fire strength) after they had cooled to ambient temperature. A schematic of these experiments is illustrated in Figure 3.1. These slab experiments (A through C) did not show tendon failure, but they did show significant stress relaxation (nearly 50%) which has an effect on the load balancing capabilities of the concrete slab – particularly in the unbonded tendon configuration. The details of the PT concrete experiments, including experimental procedure and results, can be found in [57]. The published accounts of these experiments advocate for modelling to be considered, but to date, limited attempts have been made to utilize these tests for modelling purposes, mainly due to the highly complicated nature of LITS observed in the tests which also deserves additional study.

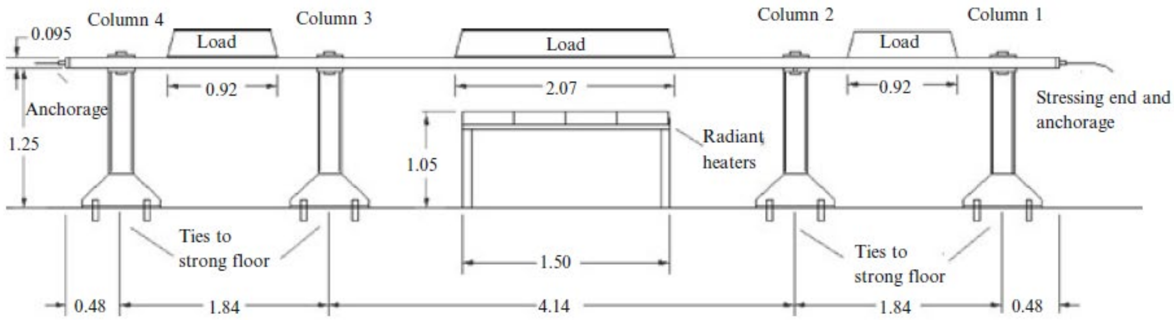

Figure 3. 1: Experimental three-span unbonded post-tensioned concrete setup, adapted from [6], exposed to localised heating under the middle span (dimensions in metres).

Recent industry reports, such as NIST 1188 [104] and ASCE Manual of Practice (MOP) [105], have called for additional research into the performance of PT concrete in fire as well as a more complete understanding of the realistic thermal boundary these structures are expected to encounter in fire. The industry consensus research needs from the ASCE MOP are highlighted in Table 3.1 and are largely adapted from NIST 1188 guidance that followed the proposed research needs identified within reference [57]. This chapter starts to address these critical research needs, focusing on a valid acceptance criterion (tendon failure and relaxation) for UPT concrete in fire against a range of realistic fires. The emphasis of this chapter is not to investigate spalling or other thermal mechanisms, such as LITS. Instead, the focus is on research needs 2 and 3 as identified in the document and Table 3.1: local and global behaviour; and analytical tools and analysis to lead to novel design tools. It is expected that future research will aim to address these needs and various organisations, such as RILEM, are currently studying these phenomena [105]. Unbonded PT concrete fire research is an important and needed area of investigation which has received little contemporary attention, in part due to its difficult to study nature. If prestressing steel tendons do fail, they have the potential for significant damage and life safety concerns, as shown in the case studies examined in [57].

Table 3. 1: Research Needs from ASCE Manual of Practice [105].

Topic	Milestone
1. Concrete Cover Spalling	a. Determine acceptable limits for spalling
	b. Quantify the beneficial effects of specific mitigation techniques
	c. Develop appropriate acceptance criteria for concrete cover spalling
	d. Develop new spalling mitigation recommendations
2. Local and Global Behaviour	a. Define acceptable limits for out-of-plumb concrete column behaviour and slab deflection, particularly when related to realistic restraining mechanisms in actual buildings
	b. Develop appropriate local and global behaviour acceptance criteria which considers the structural system performance of reinforced and prestressed concrete structures
3. Analytical Tools and Analysis	a. Develop acceptance criteria for reinforced and prestressed concrete structures under fire exposure that consider realistic fire scenarios

3.3 Properties of Prestress Steel

Currents guidelines for practitioners by the Concrete Society [106] that assess prestressed concrete structures' post-fire performance use a graphical interpretation of the remaining strength that is dependant only on the peak steel temperature reached during the fire. This guidance has been widely used in the industry [107]–[109], and is illustrated within Figure 3.2a (reproduced from [103]). The severity of the fire, including its duration at peak temperatures, is not accounted for in that guidance. Guidance is provided in a journal paper of which the thesis author is the primary author [110] which does consider the post-fire behaviour, shown in Figure 3.2b. This paper is included in Appendix D. Creep is implicitly included in these fire parameters. Relying on only the specific peak temperature for determining the remaining material strength provides a simple method for rapid estimation purposes but potentially fails to consider the effect of extended periods of high temperature exposure adequately. The peak temperatures can usually be determined by referring the heated concrete color to the guidance, which in turn is used to assess the remaining strength and load-carrying capacity. Despite its simplicity, a difficulty associated with this method is that its accuracy relies on qualitatively determining the post-fire concrete colour to determine steel temperatures [111]. Another possible method to identify the peak temperature is from a microstructural analysis of the concrete but this is time consuming. Literature reports 600°C as the most common critical temperature for prestressing steel after fire exposure, which the correlation suggests the remaining strength to be approximatively 50% of its original capacity after exposure to that temperature. Using a critical temperature approach is a common method in North America and Europe [107]–[109] and is the most convenient analysis regarding the post-fire damage assessment of prestressing steel. This method has its origins in the 1960s [112]–[114], which could be problematic since steel manufacturing has changed extensively since then. Modern fabrication techniques such as alloying have been shown to have an effect on the strength of steel with exposure to fire [103].

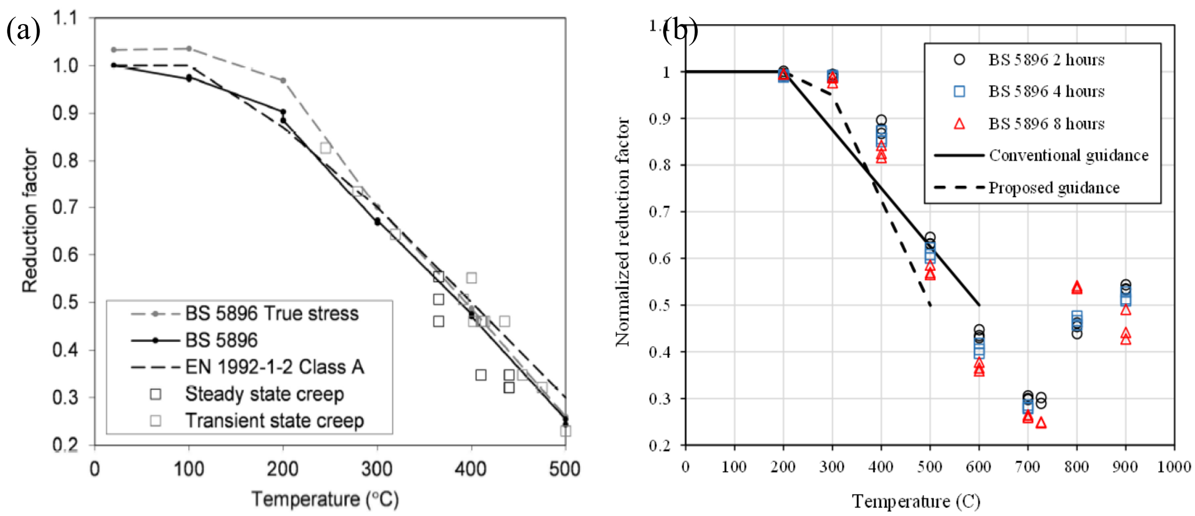

Figure 3. 2: Strength reductions with temperature for BS 5896 prestressing steel outlined in (a) EN 1992-1-2 (reproduced from [103]) and proposed in (b) Jeanneret *et al.* [110] compared to experimental results.

For potentially long thermal exposure conditions, when it is not certain that current guidance is adequate, the steel can be tested destructively. The problem in that situation is that the steel cannot be replaced or re-tensioned once removed from the structure. This leaves the structure without the beneficial effects from the pre- or post-tensioning. Therefore, a more robust method to conservatively assess the residual strength within the prestressing steel post-fire is required to ensure the safety of critical prestressed concrete infrastructure.

Spalling beneath the prestressing steel is always prevalent even with lower severity fires. This emphasizes that localized damage to the prestressing steel can have a predominant effect on the structure's strength. The important aspect is to properly assess the remaining capacity of the prestressing steel with little invasion as the condition of the structure is highly dependent upon it.

Determining the post-fire residual strength of prestressing steel using the hardness technique gives results with good accuracy, however this method is only valid up to 700°C. Beyond this point, the microstructure recrystallizes and removes the high strength effects obtained from the manufacturing process. Severe fires where the steel is exposed to fire or where spalling occurs and the prestressing steel becomes exposed should expect the steel to reach temperatures where recrystallization will occur. The colour change of concrete can be useful to determine the peak temperature reached but it is not always the most reliable. The technique to determine peak temperature from concrete colour must be conservative for it to be used.

For further discussions regarding the post-fire structural assessment of prestressing steel, see Appendix D.

3.4 Tendon Deformation Model

To begin, a thermal and tendon rupture computational model validation was performed against the previously mentioned experimental data set available in literature [57]. This consisted of a three span unbonded PT concrete flooring system exposed to localised heating (A and C denoted in Section 3.2). The analysis assumes a thermal boundary condition representative of heating conditions experienced during all three slab experiments (see [57]). There was some minor variation in heating along the slabs, therefore a representative thermal boundary condition was developed which adequately describes the adiabatic temperature-time exposure to the slabs induced in all experiments. The boundary represents a best-fit, least-squares approximation of the average of eight thermocouples used on the exposed soffit in each of these experiments (see Figure 3.3 for extent of heating). Using this heating curve and a heat transfer analysis through concrete (detailed below), temperature profiles through concrete slabs, including at the position of the tendon, can be determined.

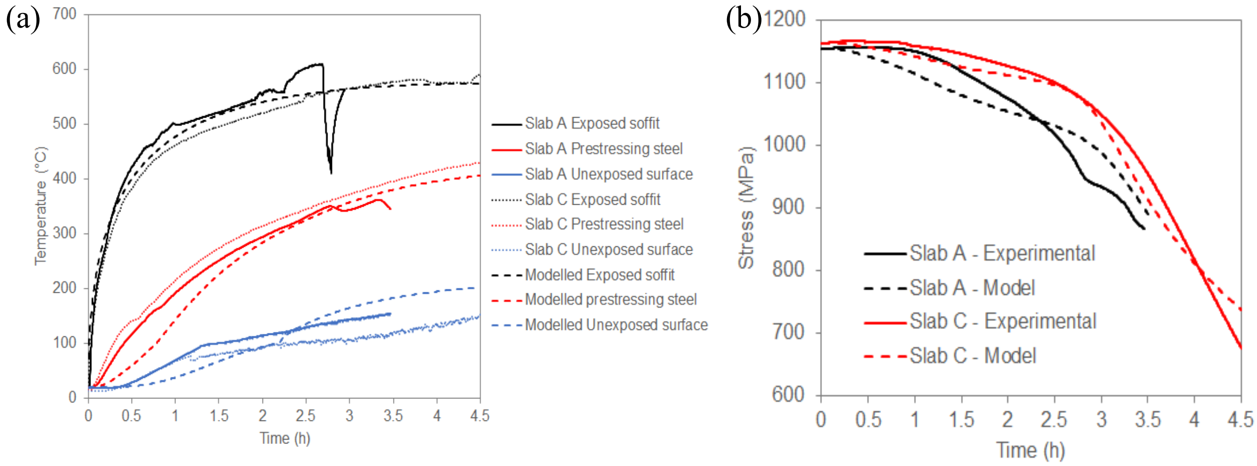

Figure 3. 3: (a) Thermal response and (b) prestressing tendon response of UPT multi-span experiments A and C (see [57]) showing good correlation between the experimental data and the model. The maximum difference between calculation and experimental results for the stress relaxation (b) is 47 MPa (5% error) for Slab A and 27 MPa (3% error) for Slab C for the first three hours of testing.

The heat transfer analysis in the concrete was performed using a one-dimensional finite difference elemental energy balance, based on fundamentals described in [115]. The input parameters considered carbonaceous aggregates, a 95 mm thick slab, and a 4% moisture content. The analysis incorporates the variations of thermal properties of the concrete and the effects of moisture due to spatial and temporal variations of temperature. The thermal properties – thermal conductivity, density, and specific heat – were calculated for the different depths within the slab for each time step during the fire exposure. This led to changing temperatures at each location with time, which affects the properties that are dependent on temperature, referred to as thermal properties. The reinforced concrete slab is assumed to be horizontally infinite, allowing for the edge effect to be ignored. At the fire-slab and slab-air (unexposed face) interfaces, the three modes of heat transfer are considered (conduction, convection, and radiation). It should be noted that the steel reinforcement is assumed to have no influence on the heat transfer behaviour herein, both for the heat transfer through the slab and the longitudinal heat transfer. This has been validated for bonded steel reinforcement but has also been shown to be very minimal in [2]. The through thickness slab temperatures were estimated and used to determine tendon temperatures (at axis distance). The temperatures obtained from the thermal modelling were compared to the experimental thermocouple temperature observations of the average surface temperature of the exposed soffit, tendon temperature, and the top of the slab at mid span in Figure 3.3a. The analysis indicated difficulties in heat transfer modelling with the moisture in concrete at the early stages. Since the heat transfer model does indicate good agreement at temperatures known to be important for creep of prestressing steel (i.e., in the range of 300°C), the thermal modelling is satisfactory for predicting the temperatures the prestressing steel embedded in concrete will experience. Using the tendon stress relaxation (which accounts for thermal relaxation and creep) and rupture modelling (degradation of strength) found in detail within the validated models of Gales *et al.* [103], the expected stress relaxation was computed for BS 5896 prestressing steel for Slabs A and C. It should be noted that other stress relaxation models and parameters exist in recent literature, however, as this work builds upon reference [57], only the parameters and material studies from that research are utilised herein. As illustrated in Figure 3.3b, the stress relaxation calculated was then compared to the tendon stress observed in the experiment.

Slab A showed a maximum difference of 47 MPa (5% error) between calculation and experimental results of stress relaxation, while Slab C showed a maximum difference of 27 MPA (3% error) during the first three hours of testing. The observed accuracy of the structural stress relaxation model is in line with that demonstrated in Gales *et al.*[103] (i.e. less than 6% error). The experimental data from Slabs A and C show little stress relaxation at the start of the experiments, whereas the structural modelling predicts clear relaxation during this period. This could be due to tendon elongation caused by thermal bowing at the start of the experiment, which would counteract (by increasing stress) stress relaxation induced by heating. However, this is not accounted for in the current stress relaxation modelling since the model does not consider structural deformation response. As shown in [57], deflection (measured as 12 mm maximum) was relatively small considering the spanning length (the length of the heated span was 4140 mm) and therefore the resulting elongation of the tendon from deflection would be expected to be small and within the error observed. The comparison between modelled and experimental tendon stress relaxation of Slabs A and C however supports the use of the utilised creep parameters from [103], allowing the stress relaxation model described to be used on the case study described in Section 3.5 and subjected to limitations described in Section 3.6.

3.5 Thermo-Mechanical Analysis of an Unbonded Post-Tensioned Concrete Structure Exposed to Fire

3.5.1 Building Case Study

The structural design of the case study considered is based on an existing PT concrete building, designed by ARUP in accordance with UK building standards. Figure 3.4 illustrates the 2-storey structure and Figure 3.5 illustrates its structural layout. It should be noted that the analysis is hypothetical based on several changes in the building's design.

Figure 3. 4: Exterior corner of the two-storey building used for the case study. This is a post-tensioned concrete structure designed by ARUP with over 100 prestressing tendons of various lengths. Photograph provided by ARUP.

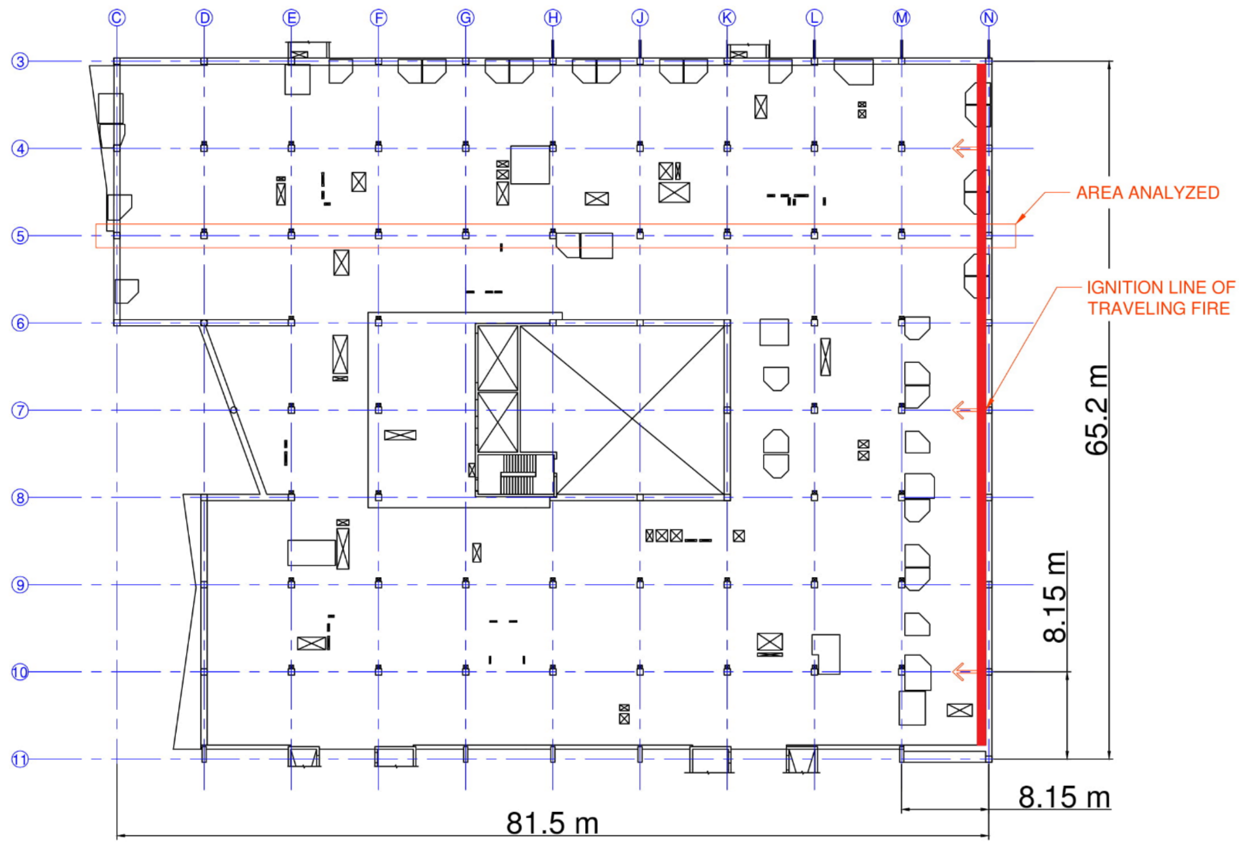

Figure 3. 5: Structural layout for the case study building, showing column grid spacing of 8.15m. The ignition line for the travelling fires analysed is shown on the right, as well as Grid line 5 along which the tendon length is varied (note the prestressing grid is not as designed for simplicity within this case study).

The structure has column-to-column spans of 8.15 m, with over 100 prestressing steel tendons varying in length from 6 to 32 m. The height of the compartment to the soffit is 3.6 m. Figure 3.6 shows the parabolic tendon profile within the typical 250 mm thick slab, which has a 35 mm tendon axis distance at the supports and mid-span. To address the needs and limitations of this study, certain design aspects of the structure have been modified from those of the actual structure. The slabs are not representative of a real building; however, they do provide value in the development of research tools to investigate various physical responses.

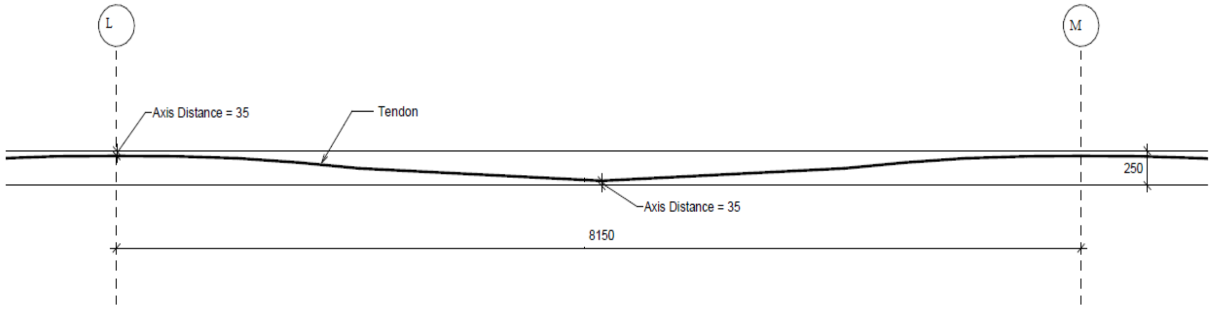

Figure 3. 6: Cross section of the slab between Grid M-L, along Grid line 5, showing the concrete cover and parabolic prestressing steel tendon (dimensions in mm). Obtained from [57].

The two key variables in this study are the design fires and the tendon length. For each analysis, a fire origin from the west side of the structure (right façade of the structure shown as Grid N) was considered unless the fire was specified as uniform. The tendon length was varied in each analysis, originating from Grid N and projected the tendon length along Grid 5 for each analysis. The study considered eight simulated tendon lengths for each design fire. The tendon length was defined as starting at 8.15 m and using the column-to-column spacing (8.15 m) as the interval. The longest tendon considered was 65.2 m, which was considered as an upper bound limit for PT construction. Two tendon types (I and II) as defined in Section 3.5.3 are considered. A total of 128 fire-tendon fire scenarios were analysed and discussed herein (Section 3.5.2 discusses the design fires used). Table 3.2 illustrates these scenarios.

Table 3. 2: Prestressing steel tendon analysis scenarios considered.

Fire Scenario	Tendon type	Tendon Starting Grid	Fire Starting Grid	Number of tendon lengths analysed [a]
Standard (ISO 834)	I and II	N	Full compartment	16
Parametric (Short)	I and II	N	Full compartment	16
Parametric (Long)	I and II	N	Full compartment	16
Travelling fire (2%)	I and II	N	N[b]	16
Travelling fire (5%)	I and II	N	N[b]	16
Travelling fire (20%)	I and II	N	N[b]	16
Travelling fire (40%)	I and II	N	N[b]	16

[a] Eight tendon lengths of increasing size were considered for each tendon type.
[b] Additional analyses were done starting on Grid M but resulted in no significant changes and are therefore not considered.

There are limitations that need to be considered prior to the presentation of the case study's results. First, it was not attempted to investigate the thermal-mechanical response of the concrete slab. Very limited progress has been made in literature on this subject owing to a lack of available experimental programs to allow its validation. Transient thermal straining, more specifically Load Induced Thermal Strain (LITS), has received some attention by researchers in recent literature [116]. This straining mechanism appears to be important in describing the deformation response of PT concrete slabs which thereby make performance criteria difficult to establish [104], [105]. Tendon rupture and stress relaxation as a function of tendon length and fire exposure is the current focus of this investigation, analysed through the case study herein.

3.5.2 Design Fires

The case study structure presented herein was thermally evaluated for a range of design fires, including the uniform ISO 834 standard fire, uniform parametric fires and non-uniform travelling fires, to determine the concrete's thermal response as a function of time (see Table 3.2 for configurations considered). All fires were selected qualitatively based on experience from previous studies to reflect a typical design process and a broad range of what the compartment may experience. The heat transfer analysis of structural members was calculated using the nonlinear finite difference calculation as defined in Section 3.4. The inputs to the calculations, such as a moisture content of 1.5%, are standard values obtained from the Eurocodes [70]. It was assumed that explosive spalling is unlikely to occur (though should always be of attention) as the XC1 class is considered with moisture content less than 3% and the concrete strength is below 55 MPa, in accordance with EC2-1-2 [117]. The most critical fires that may cause tendon rupture, based upon region specific material usage, are identified using the validated and highly conservative region-specific tendon rupture models from [103]. These models are further described in Section 3.5.3 and are used to identify at which temperature the tendons are expected to rupture.

Seven varying design fires were considered in this analysis to cover a range of different structural behaviour: a short hot parametric fire, a long cool parametric fire, and four travelling fires, and the standard fire (ISO 834). Although the standard fire is a prescriptive exposure that is not representative of reality, it was included due to its frequent use in design and, more importantly, to provide a benchmark for comparison. The design fires were selected based on what is considered current practice in the industry, based on the UK standards PD7974-1 and -3 [78], [79]. For all design fires, fire load density and heat release rate per unit area were taken for office type accommodation in accordance with EN1991-1-2 (511 MJ/m^2 and 290 kW/m^2 respectively) [70]. As Rackauskaite *et al.* [93] have shown, travelling fires of different speeds can lead to different thermal and structural responses. As a result, for the purposes of this study, four representative travelling fire scenarios were qualitatively considered of two slow, a medium and a fast travelling fire (2%, 5%, 20% and 40% fire sizes with respect to the compartment size respectively) to ensure a range of responses. A flapping angle of 6.5° was adopted [93]. Figure 3.7 illustrates the time-temperature curves for the four travelling fires across the entire structure's floor. The two parametric fires used were determined to be representative of different structural behaviour, with a hot-short fire and a long-cool fire, found by varying the available ventilation as discussed by Lamont *et al.* [118]. The thermal exposure of the two parametric fires are shown in Figure 3.8. The standard fire was limited to a two-hour thermal exposure, as this would be the typical fire rating prescribed for this system.

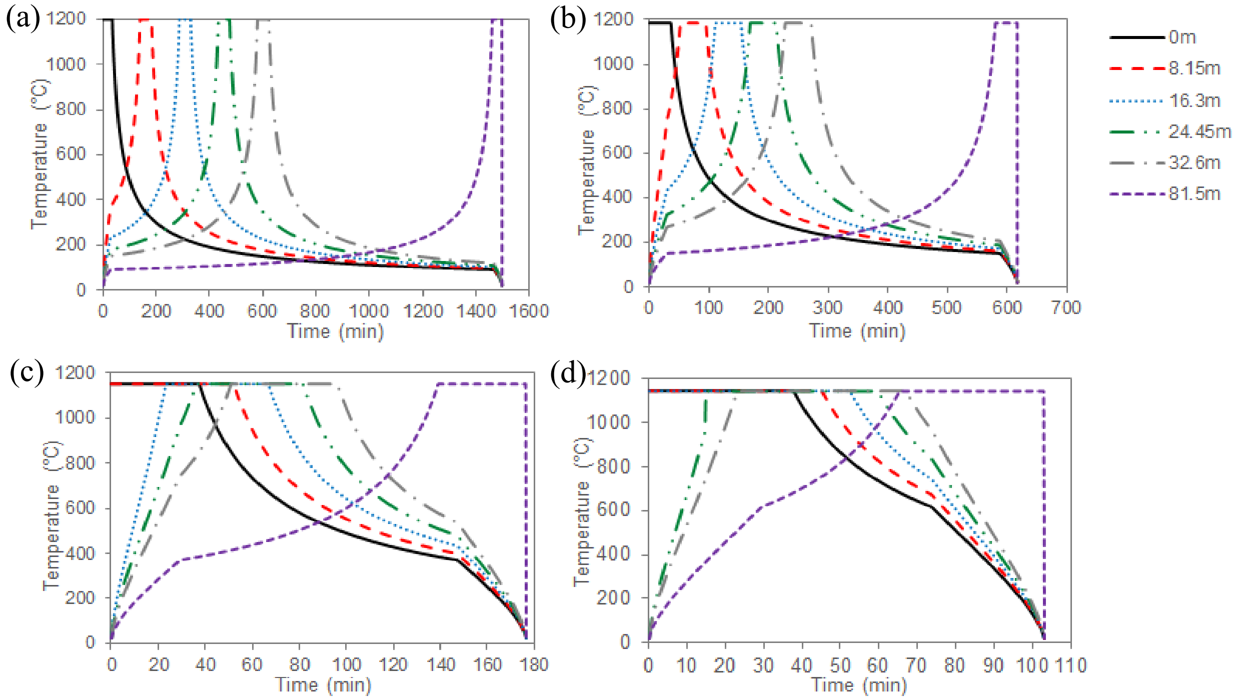

Figure 3. 7: Thermal exposure across gridline N-C for the four varying sizes of travelling fires examined: (a) 2%, (b) 5%, (c) 20%, and (d) 40%. The graphs illustrate the thermal exposure at regular intervals in the compartment.

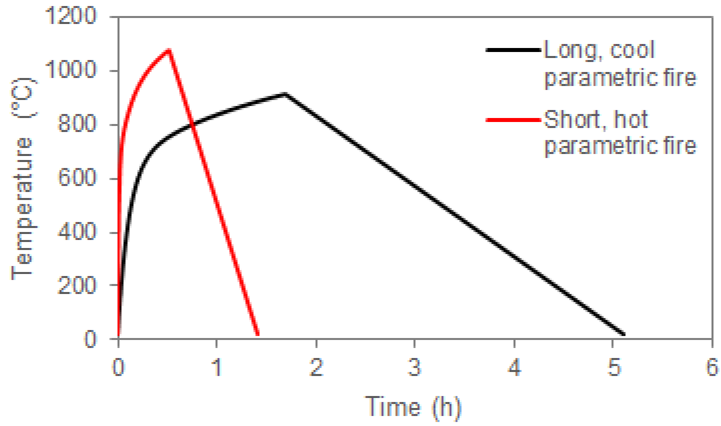

Figure 3. 8: Thermal exposures of the two parametric fires: a short, hot fire and a long, cool fire. These were found by varying the ventilation until the two extremes of parametric fires were found.

3.5.3 Tendon Analysis

After the thermal boundary conditions (from design fires) and concrete temperatures were calculated (from the same one-dimensional nonlinear finite difference calculation method introduced in Section 3.4), the corresponding tendon temperatures were extracted from the thermal analysis. These temperature profiles were then inputted into a stress relaxation model developed and described in reference [103]. The model was developed by calculating the thermal creep strain increase that occurs when the steel is exposed to a changing high temperature during a set time interval over a set length of the steel. The model then sums the strain increases over the length of the tendon to calculate the overall relaxation of the unbonded tendon stress. Two types of steel were analysed: Steel I (fabricated according to BS 5896) and Steel II (fabricated according to AS/NZ 4672). The prestressing stress and strength analysis considered both these steel types to assess whether region specific influences in manufacture may control the response of the unbonded prestressing steel. Once the prestressing steel stress relaxation was calculated using thermal properties from [103], significant differences in behaviour emerged with respect to relaxation and the introduction of the travelling fire as the design thermal boundary. Figure 3.9 and Figure 3.10 briefly illustrates the response of one common 32.6m long unbonded prestressing steel tendon exposed to the travelling fires examined. Strength reduction due to increasing temperature was calculated using the recommended reduction factors in EC2-1-2 [117], shown within Table 3.3.

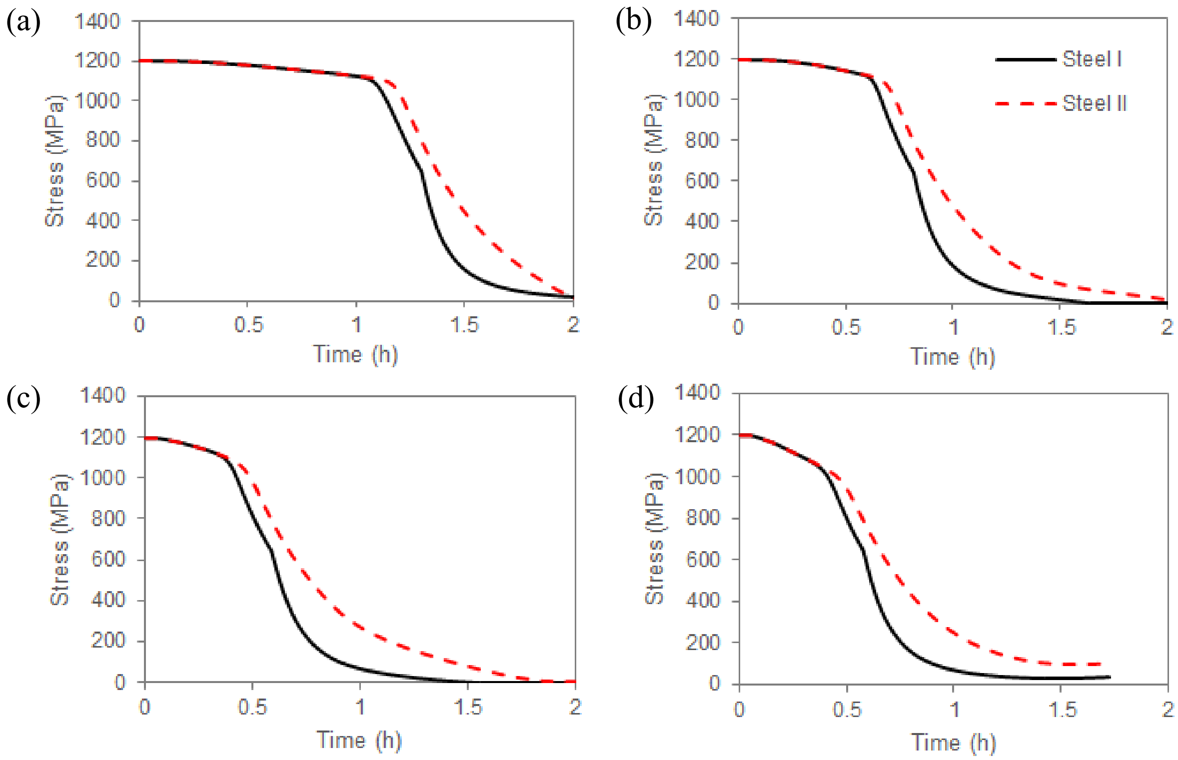

Figure 3. 9: Tendon stress relaxation model of a 32.6m tendon exposed to the four travelling fires: (a) 2%, (b) 5%, (c) 20%, and (d) 40%. This figure illustrates that Steel II (fabricated according to AS/NZ 4672) experiences less stress relaxation over time than Steel I (fabricated according to BS 5896).

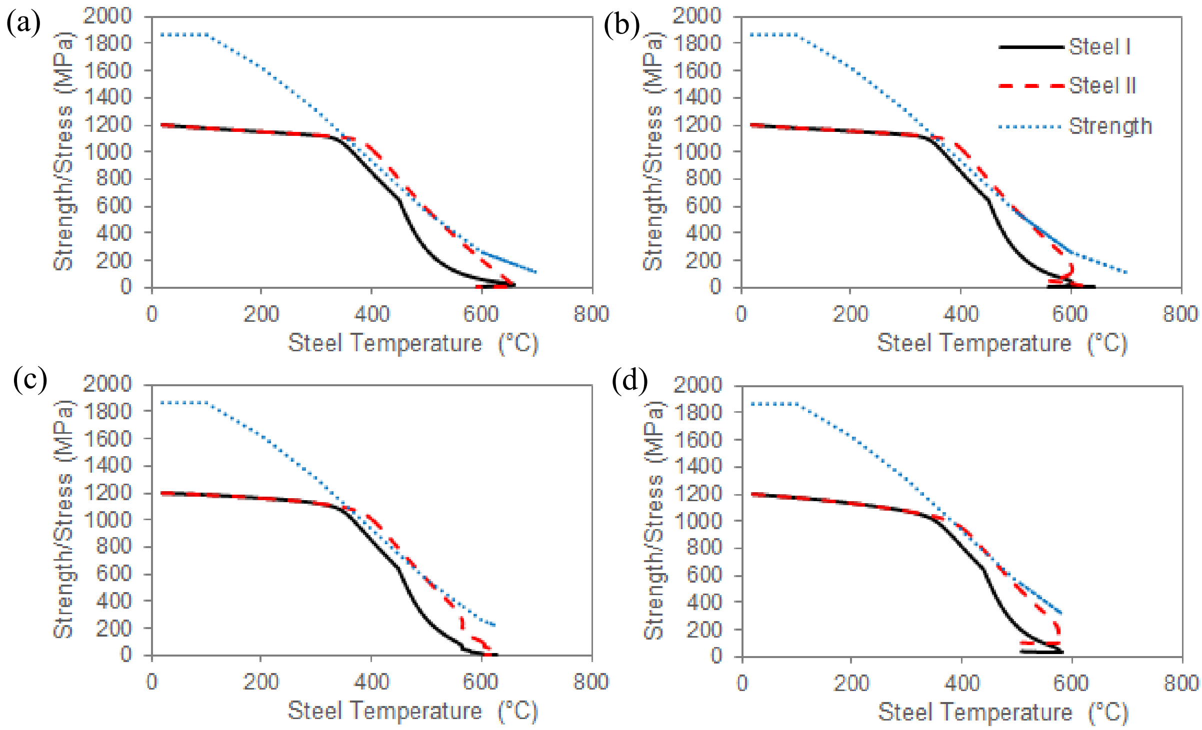

Figure 3. 10: Tendon strength reduction with temperature of a 32.6m tendon exposed to the four travelling fires: (a) 2%, (b) 5%, (c) 20%, and (d) 40%. This figure illustrates that Steel II (fabricated according to AS/NZ 4672) would fail due to tendon rupture (where the stress surpasses the strength of the tendon).

Table 3. 3: Strength reduction factors for cold worked prestressing steel tendons at elevated temperature, given in EC2-1-2 [117].

Steel Temperature (°C)	**Strength Reduction Factor**
20	1
100	1
200	0.87
300	0.7
400	0.5
500	0.3
600	0.14
700	0.06
800	0.04
900	0.02
1000	0
1100	0
1200	0

The stress relaxation results first imply that Steel II, in all cases, has less stress relaxation than Steel I. Figure 3.11 illustrates the comparison of this for a 32.6m tendon. When various travelling fires are considered, they seem to converge by two hours to a below-acceptable stress state regardless of the make of steel (I or II). The acceptable stress relaxation was assumed to be 50%, in this case 600 MPa (defined by stress relaxation without collapse observed in reference [57], this value should be used with caution as describe in Section 3.5.4). For most cases, the stress relaxation calculated due to travelling fires is more significant than a uniform fire exposure (parametric and standard) for the unbonded prestressing steel tendon. This is likely as the travelling fire leads to longer heating durations and therefore higher steel temperatures, but further studies are required to assess the influence of travelling fires in experimental reality. Based on the acceptable stress state described, the temperature of the steel which would cause this stress state can be identified. Figure 3.12 illustrates the temperature at which stress failure occurs with varying tendon lengths.

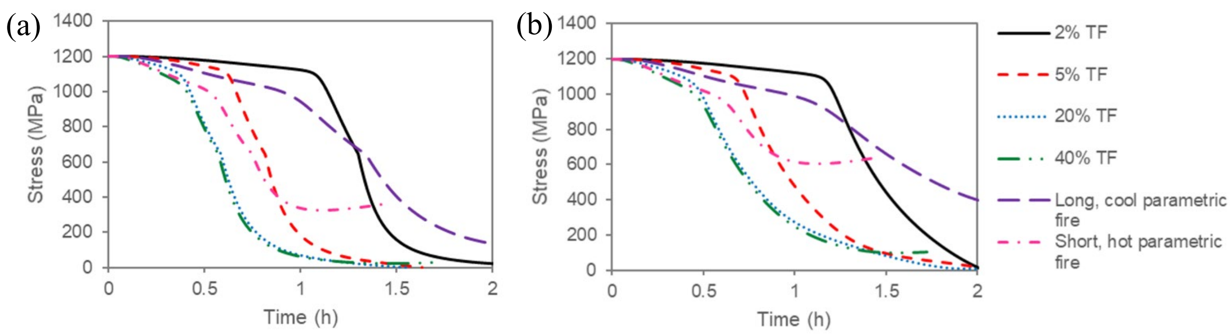

Figure 3. 11: Tendon stress relaxation model of a 32.6m tendon for (a) Steel I (fabricated according to BS 5896) and (b) Steel II (fabricated according to AS/NZ 4672) under the various design fires examined, illustrating that travelling fires cause greater stress relaxation. This figure also shows that for all cases except the short, hot parametric fire for Steel II, the tendon would fail due to stress relaxation (<600MPa).

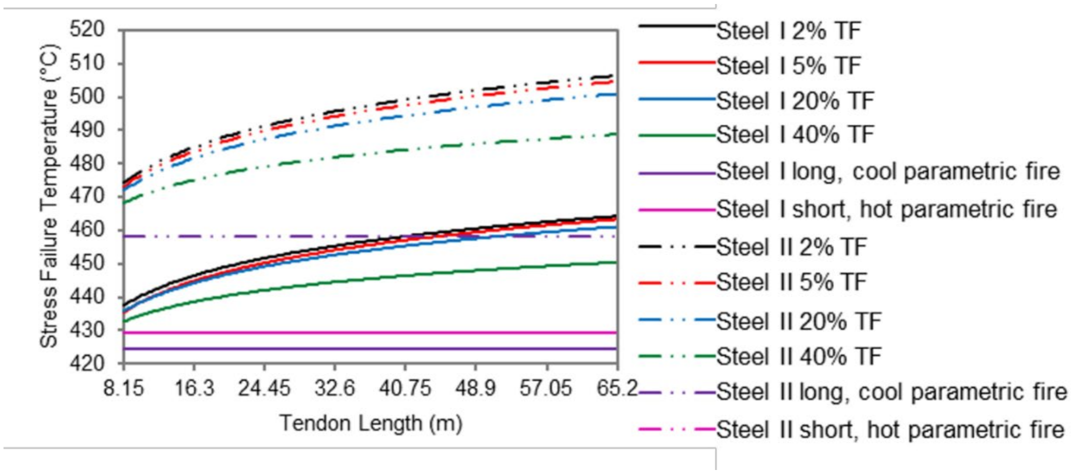

Figure 3. 12: Contour plots showing the temperature at which varying prestressing tendon lengths experience a stress failure (stress in tendon < 600MPa) for all design fires considered. The figure illustrates that the steels would fail at temperatures higher than the critical design temperature specified (350°C for the Eurocode and 427°C for the North American standard).

When tendon strength reduction is considered, the more onerous fire emerges as the small travelling fire (5%) with respect to tendon rupture. In strength reduction, tertiary creep is simplified in the calculations and rupture is implied to occur during secondary creep when tendon stress exceeds time dependent strength. The omission of tertiary creep in the calculations produces conservative results since only considering secondary creep decreases the stress relaxation and therefore increases the stress level and predicts tendon rupture sooner. To fully define strength failure including tertiary creep, further research is needed, however, the creep model has been shown to be accurate in [103]. Rupture is not predicted to occur for Steel I but is predicted for Steel II between 342°C and 400°C. The strength reduction due to increasing temperature was calculated using the recommended reduction factors given in Table 3.3 and in EC2-1-2 [117], which are considered typically conservative with respect to available literature [103]. However, Robertson *et al.* [119] indicated very significant strength reductions as a function of heating time (even when specimens had no load during heating). It is prudent that, if a long travelling fire is considered, the corresponding strength reduction should also be considered more carefully. This is because, as prestressing steel is heated for a longer period at the same elevated temperature, it can lose additional strength. Figure 3.13 shows the temperature at which tendon rupture is assumed to occur with varying tendon lengths.

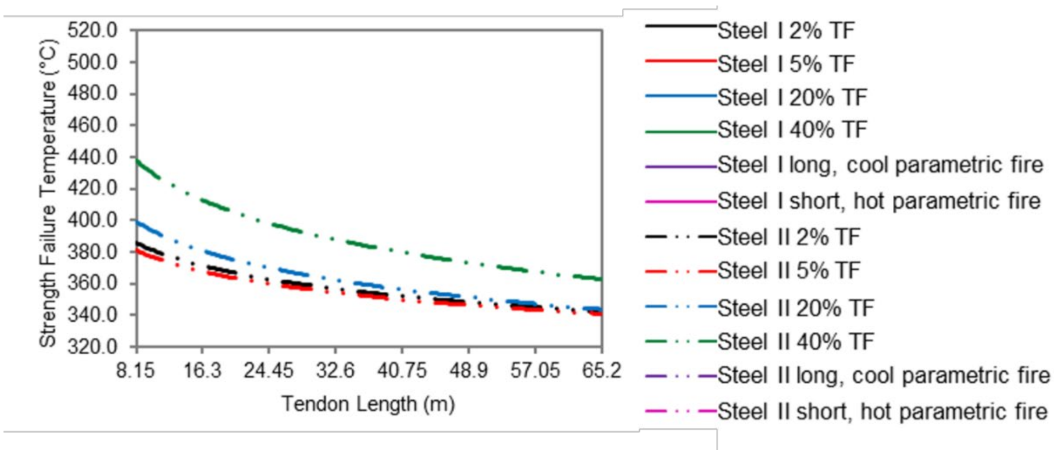

Figure 3. 13: Contour plots showing the temperature at which varying prestressing tendon lengths experience a strength failure (where the stress surpasses the strength of the tendon) for all design fires considered. The figure illustrates that Steel II would fail prior to reaching the Eurocode critical design temperature (350°C) for tendon lengths greater than 42m, and would fail to meet the North American standard (427°C). Steel I does not fail due to strength failure.

Additional analyses were performed with the tendons starting on Grid M (instead of N) and the fire igniting along Grid N for travelling fires (same as previous analysis). This was done to examine the effect on tendons being offset to the start of the fire, which could occur in a realistic structure. The results, however, did not vary significantly, with no differences for the strength failures and differences of less than 5°C for stress failures. The uniform fires did not result in any differences since the location of the tendon has no impact on the thermal exposure it will receive. Since the results are not significant, these analyses were not included in Table 3.2 and any further discussion.

3.5.4 Acceptance Criterion for Structural Design

Figure 3.12 and 3.13 can be used for a simplified basis of establishing an acceptance criterion for cases similar to the one examined. These figures identify the temperatures which would cause failure of the structural system, using the assumptions that stress failure occurs when 50% stress relaxation is reached, and tendon rupture occurs when the stress exceeds the strength of the tendon. The criterion can be appropriately defined by the stakeholders. A 50% stress relaxation was assumed as the definition of stress failure as this has been shown not to cause collapse of a slab within experimental tests [57]. It is anticipated in reality that the value may be considered higher, but the practitioner can easily follow the methodology herein to establish a separate set of contours for a higher allowable stress relaxation state.

Figure 3.13 illustrates the strength failure temperature versus the length of strand, showing that travelling fires are more critical than parametric and uniform fires (which did not cause tendon rupture) for this specific case study and the design fires examined. Only Steel II experienced tendon rupture, and only when exposed to travelling fires. Figure 3.13 illustrates that, as the tendon length increases, the most critical failure temperature decreases. This is expected as the fire becomes less uniform, but the defining critical temperature can drop below 340°C. When designing PT concrete slabs, the Eurocode uses 350°C as the critical temperature which would cause failure of the slabs while the North American guidance uses 427°C [120]. The concrete design is done so that the steel tendons only reach these critical temperatures once the specified fire rating is reached. Figures 3.12 and 3.13 allow the designers to rationally decide the longest tendon length that should be supplied using the contours created (based upon the criterion of 50% stress loss and Eurocode strength reductions). It is subsequently advised that if the critical temperature of 350°C is used for prestressing steel, the maximum tendon length specified for Steel II should be 42 m. If a critical temperature of 427°C is assumed, as in North American standards [120], the maximum tendon length is less than 8 m due to tendon rupture. Since Steel I does not rupture, the standard chosen would be of no consequence.

For this specific case study and the design fires examined, a slow travelling fire (5%) is the critical design fire that is required for tendon rupture. The result is expected as the fire leads to longer durations that the concrete will be exposed to heat. While a 2% travelling fire would lead to a longer thermal exposure, the concrete is exposed to the near-field (peak temperature) for a very short period of time and allowed to cool in the far-field for longer durations.

3.6 Limitations and Gaps in Literature

The results obtained as part of this study are only the first steps to defining the critical design thermal boundaries for UPT concrete flat-plate slabs. Additional research is required, which will require additional tendon modelling to confirm. These include but are not limited to: travelling fires moving perpendicular to the current analysis since the tendons span multi-directional (east west, north south, and diagonal); tendons that are multi-banded; bond type; various tendon profiles; and sensitivity of concrete properties. The modelling described herein utilizes the iTFM methodology, developed in 2015 by Rackauskaite *et al*. [71]. This methodology has been recently been updated [121], as well as the availability of other travelling fire methodologies that incorporate different variables and methods to calculate thermal distribution within the compartment. These could be considered within further studies. The undertaken structural modelling does not consider the thermal-mechanical relations of load induced thermal strains (LITS), and thermal gradients which invoke complicated deflection mechanisms or load shedding from tendon failure. All of these research advancements are beyond the scope of the current study but necessary to consider when formulating generalized design considerations for highly complex prestressing steel.

More experiments are needed to fully understand the tendency of spalling and its effects on the tendons within the slab [122]. Prestressing steel tendons, compared to mild steel reinforcement, can experience more damage under high temperature. If spalling were to occur, the tendons would be exposed directly to the severe heating and the significance of such heating is still unknown. A more complete understanding is needed to inform a more in-depth tendon deformation model.

Further follow-up studies from this work could consider the consequences to the reduction in flexural and shear capacity of UPT concrete slabs as a result of localised tendon rupture. This may also require further experiments to validate, particularly in defining the amount of required bonded non-prestressed reinforcement to prevent collapse. Preliminary insight into stress relaxation and tendon rupture effects on shear and flexure can be found elsewhere in Gales *et al.* [123].

Future research will inform practitioners how to structurally design these assemblies to prevent tendon rupture and improve upon probabilistic fire design approaches for this structure type. The research can lead to the needed acceptance criterion to help enable objective- and performance-based strategies, meeting ASCE MOP goals. The current tendon stress relaxation model has only been validated for unbonded prestressing steel, using the experiments in [57]. The research within, therefore, cannot be applied to bonded PT concrete. Once a model is developed and validated for bonded prestressing steel, the same method described herein could be used.

Another key research need is additional large-scale experiment validation of fire dynamics in large compartments. This would allow for an improved modelling of the thermal boundary and subsequent heat transfer. Those efforts are underway by multiple researchers, which will allow for further probabilistic investigations.

The research herein has shown that the quality of steel is very important. The stress relaxation of Steel II was shown to be less than Steel I (AS/NZ 4672 and BS 5896 respectively). The reason Steel II performs with less relaxation is because the stock considered in reference [103] had a sizably larger concentration of chromium than the stock of Steel I, which can improve creep properties [103]. The origin of the steel is inconsequential; it is the quality of the steel that is important and should carefully be checked should a performance-based fire design and analysis be performed on a UPT structure. There is no guarantee that this addition of chromium was intentional or if it is consistent. Hence, it is recommended to lean towards conservatism to prevent an extensive and expensive material study performed for every performance-based design.

The discussion regarding concrete design within this chapter and Chapter 2 highlights the need for the development of an annex within the CSA A23.3-14 Design of Concrete Structures. Both its steel and timber counterparts (CSA S16-19 and CSA O86-19, respectively) currently outline fire design approaches within annexes that can be used for that specific material, promoting the use of alternative solutions to practitioners. The development of an annex for concrete design would allow for methods for comparing alternative solutions to acceptable criterion to be explored, guiding practitioners in their structural fire design. The research presented herein only captures the behaviour of UPT concrete in fire, therefore, the development of an annex would require additional research to develop acceptance criteria and methodologies for safe structural fire design of concrete structure. All three material standards should also include hybrid and composite guidance, currently preliminarily shown in the steel design code CSA S16-19 for concrete steel composite structures.

3.7 Summary

This study has built upon the previous endeavours in PT concrete research. iTFM was used for the first time on a UPT concrete structure to define critical design thermal boundaries and resulting unbonded tendon behaviour. A comparison of the effect of uniform and travelling fires for UPT structures is novel since it has not been reported in literature before. The Gales *et al.* stress relaxation model [2] was compared and validated to real test data [57], which, once further validated with additional experimental data, could then be used in a novel fashion to assess a realistic building floor plan as opposed to a purely hypothetical structural configuration (where structure size, slab thicknesses, and span dimensioning may not be consistent with as built reality) to develop acceptance criterion as defined by the ASCE Manual of Practice [105].

Designers are in need of simple analytical tools which can enable rational fire design. When a family of fires is considered, it allows for the possibility to optimize the safety of a PT structure in order to facilitate the specification of a maximum tendon length that will not relax nor rupture. However, UPT concrete structures are of high complexity and require additional development of these analytical tools.

The results of the study establish the first steps towards the definition of the critical design thermal boundaries for UPT concrete flat-plate slabs. This study outlines the methodology that can be used to develop these critical boundaries for different structures. For this specific case study and the design fires examined, a slow travelling fire (5%) is the critical design fire that is required for tendon rupture. The result is expected as the fire leads to longer durations that the concrete will be exposed to heat. The analysis of this case study suggests that Steel II would fail due to tendon rupture between 342°C and 400°C. For this reason, a maximum tendon length of 42m can be considered when using steels that exhibit similar behavioural characteristics as Steel II in similar sized compartments, but should be used with caution when considering the limitations of this study. Steel I showed no vulnerabilities to strength failure (tendon rupture) but illustrated severe stress relaxation potential. In relation to stress relaxation, uniform fires are more critical for steels that exhibit the characteristic of both prestressing steels (I and II) compared to travelling fires. It was found that, by simulating a range of traditional and travelling fires, the tendon rupture temperature was higher than the critical temperature specified in the Eurocode when Steel I was utilised (minimum temperature of tendon rupture being 425°C to a critical temperature of 350°C), from which it can be assumed there is no risk of tendon rupture.

The development of the critical thermal boundaries allows for simpler and safer design for UPT concrete exposed to fire. The research herein has shown that the quality of prestressing steel is imperative to consider in the design, and that to ensure the most severe fire loads are considered, travelling fires must be included in the design fires. The localised heating effect and longer duration of slow travelling fires create more onerous conditions within the structure. Further research is needed to develop an annex within CSA A23.3-14 Design of Concrete Structures which will allow for greater use of safe and resilient alternative solutions in concrete Canadian design.

Chapter 4:
Connections in Fire

4.1 General

Steel structures are very common and the behaviour of their beams and columns during fire events has been extensively studied. The behaviour of steel beam-to-column connections within these structures under thermal loading, however, is not well understood. When exposed to a fire, the members within steel structures expand during heating and contract in cooling, inducing large forces upon the connections. As discussed in Chapter 2, how the structural elements are restrained has an effect on their performance during fires. Restraint is determined by the surrounding structure and the types of connections used, which the latter determines how the forces are transmitted from the beams into the columns. Performance-based design requires innovation by steel fabricators to create technologies that will be able to implement optimized design solutions. To ensure resiliency, steel connections, such as beam-to-column connections, must be able to withstand the loading that would be induced due to fire. The increasing demand from practitioners to create resilient critical infrastructure that can resist thermal loading has created a need for steel manufacturers to improve their connection design technologies. The behaviour of beam-to-column connections under thermal loading is currently not well understood, limiting the current design to being overly conservative rather than using a performance-based solution [124]. Previous collapses of steel structures have shown that connections are critical to structural integrity. The research discussed within this chapter was conducted in collaboration with Benson Steel Ltd, a steel manufacturer in Canada, and ARUP, an international practitioner that designs many iconic steel structures. It has been identified that there is a lack of understanding regarding steel connections, especially when the connecting beams are non-uniformly heated as would occur in a real fire [125].

Accidental fires observed have demonstrated that the global behaviour of structures can vary from small-scale or single element tests currently used in industry to determine fire resistance ratings. For example, in 1990, a fire broke out in the fourteen storey Broadgate building in London, UK while it was under construction [126]. The intensity of this fire led to the belief that the structure would collapse, but instead, there was no structural failure, and the floor slab did not lose its integrity. This behaviour was attributed to the composite floor and beam system, and the possible load distribution between the weakening members. However, there was little experimental data available regarding full-scale structures to compare with this event. This motivated several series of tests to be completed at the Building Research Establishment's (BRE) Cardington Laboratory in the UK [27]. Three sets of large-scale testing were performed in this former airship hangar on multi-storey steel, concrete and timber structures. The tests evaluated different compartment linings, fire load types and ventilation. At the same time, the tests were used to better understand the global behaviour of a structure exposed to a severe fire event.

Experimental data is necessary for the calibration and validation of computational modelling techniques used in structural fire design. The experiments planned as part of this research project begin to address the need for large-scale experiments to validate complex and detailed modelling of structures under thermal exposure. Real-scale compartment testing is not frequently performed because of its costly and time-consuming nature. It does have its advantages as it allows real fire scenarios to be tested while understanding how the entire structure would interact under such loading. It is more common, however, to test single elements within a furnace. This is a simple and standardized process; however, this can be inaccurate as the boundary conditions and the interaction between elements are neglected. As discussed within Chapter 2, it is important to consider the concept of "consistent crudeness" when designing and undertaking fire experiments. The experiments undertaken as part of this chapter were designed following this concept, using a localised pool fire and a sub-frame assembly.

This chapter aims to begin addressing the need for the understanding of steel connections in fire. Studying the underlying mechanisms (for example, heat transfer, beam deformation, connection movement) that influence the behaviour of steel connections during non-uniform heating would facilitate the improvement of connection design in order to create resilient critical infrastructure that can resist thermal loading. This chapter aims to produce preliminary design guidance and a conceptual test framework which will ultimately lead to a steel connection computational toolkit that will account for all underlying mechanisms, allowing efficiency and resiliency for steel connection design.

4.2 Literature Review of Connections in Fire

The design of steel connections in Canada is typically done with the main goal of resisting the vertical reactions transferred from the beams to the columns. It is often assumed that connections will be strong enough for fire based on a typical design since it is assumed that the connections will not reach the same high temperatures as the beam and will heat up more slowly. This, however, has been shown to be inaccurate by real fires like those seen in the World Trade Centre collapses (1, 2 &7), as well as large scale experimental testing at Cardington [27]. These fires highlighted the importance of designing connections for fire loads, since connections are crucial to the robustness and structural integrity of the buildings, yet they undergo a complex loading when exposed to fire. The Cardington tests demonstrated that the behaviour of single element testing differs significantly from full structure tests [27]. This is due to factors such as load redistribution and the thermal restraint imposed by the surrounding structure.

There has been limited experimental studies into the behaviour of beam-to-column connections under fire exposure, mainly due to the high cost of fire tests. The majority of the research is focused on modelling, using the limited experiments as forms of validation [127]–[129]. A large portion of the previous fire tests, dating early 2000s and earlier, were focused on understanding moment-rotation characteristics of end-plate connections [130]–[137]. The largest, in compartment size, experimental study utilized the Cardington steel structure to perform a large-scale test. This experiment, reported by Wald *et al.* [36], examined the temperatures selected structural elements would reach using a fuel load of 40 kg/m^2 of wooden cribs across the compartment. The connection temperatures, including bolt temperatures, were measured, illustrating relatively uniform temperatures within the connections that followed a similar trend but were lower than the temperatures experienced at the midspan of the respective beams. The structure did not experience collapse, however some local buckling and fractures occurred, some during the cooling stage. Details about the connection forces or specific behaviours were not discussed. This experiment was used to validate and guide research by Selamet and Garlock [138], which modelled in ABAQUS the interactions between connection structural elements using the experimental compartment temperatures for three types of shear connections: single angle, double angle and shear tab.

An experimental program, leading to modelling, was undertaken in Taiwan by Mao *et al.* [139] to examine steel moment connections. The experiments were conducted in two phases on a steel H-beam to H-column configuration, with shear tab connection, set up within a furnace. The first phase of the experiments exposed the setup to constant temperatures (550°C and 650°C) while loading the beam end until failure criteria were reached (maximum deflection of 500 mm or rapid change in rate of deflection). The second phase held the load on the beam end constant (400 tons or 3924 kN) and exposed the setup to the ISO 834 standard fire curve until the previously outlined failure criteria were reached. The study was then used to validate a model in ANSYS, which then allowed for the examination of various parameters that affect the behaviour of connections.

The National Institute of Standards and Technology (NIST) in the United States has conducted experiments on long span steel-concrete composite floor beams, which allowed for data regarding shear connections to be captured [140]. This series of four experiments on 12.8 m beams aimed to understanding the behaviour and failure mechanisms of this structural systems when exposed to full compartment fires. Shear tabs and double angle connections were examined. The steel beams and connections were protected using sprayed fire-resistive materials to meet the US design standard of a 2-hr fire-resistance rating. The assemblies were realistically loaded at six points and thermally exposed using natural gas burners, following a repeatable burn described as "compartment fire conditions with the heat-release rate of 4,000 kW" [140]. Strain gauges were placed on the columns to measure the axial loads created by the thermal exposures, as well as thermocouples along the steel deck, beam and connections to measure the temperature response of the assembly. The axial loads at the beam ends were reported up to values of around 1000 kN.

The majority of the remainder of the research on steel beam-to-column behaviour in fire was undertaken by the research team at the University of Sheffield with some collaborations with the University of Manchester. Two major experimental research programs were undertaken by these teams, which was summarised by Wang *et al.* within [141]. The first program comprised of isolated connection tests at high temperatures, detailed for the shear tab (or fin plate) connection by Yu *et al.* in [142]. Other connections are discussed in [143]–[145]. The experiments, conducted within a 1 m^3 electrically heated oven, heated the test setup to fixed temperatures (450°C, 550°C and 650°C) and then created tensile loading on the end of the beam to induce large rotations within the connections being tested. Three loading conditions were examined, created by changing angles of the loading system, inducing axial tension, shear and bending moments. The connections examined included fin plates (shear tab), partial depth endplates, web cleats (angle), and flush endplates. The second program undertook fire tests on structural sub-assemblies [141], [146]. The experiments investigated the effect of two column sizes and five different types of connections (those tested in the first program as well as extended endplates) on a simple 2 metre, four point loaded beam between the restrained columns. Thermocouples and displacement transducers were used to capture temperature distribution and movement. The axial force developed within the beams were also recorded throughout the fire exposure. An additional test series was conducted to help develop calculation methods to predict connection temperatures [147], [148]. The detailed finite element modelling of all of these research programs were also undertaken [143], [144], [149], [150]. The full research undertaken by this team until 2012 was summarized by Burgess *et al.* [151].

As discussed within Section 2.2.2, it is important to consider the experimental setups within research on the basis of consistent crudeness. Table 4.1 presents the discussed literature and highlights where each study would fall within the 'Crudeness Framework' by identifying the structural scale and thermal exposure used.

Table 4. 1: Relevant literature on steel connections in fire, placed within the 'Crudeness Framework' discussed in Section 2.2.2

Study Title	**Author(s) (Year)**	**Connections**	**Experimental/ Modelling**	**Structural/ Modelling Scale (1-5)**	**Thermal Exposure (1-8)**
Experimental investigation of the behaviour of fin plate connections in fire	Al-Jabri, Khalifa Saif, et al. (2006)	Flush end-plate bolted	Modelling	Partial elements (1)	Transient linear temperatures (1)
The behaviour of steel and composite beam-to-column connections in fire	Wald, F, et al. (2006)	Cardington structure – Single plate	Experimental	Real structure (5)	Parametric fire (4)
Finite element modelling of steel fin plate connections in fire	Sarraj, Marwan, et al. (2007)	Fin plate	Modelling	Partial elements (1)	Parametric fire (4)
Experimental study on flexible end plate connections in fire	Hu, Ying, et al. (2008)	Flexible end plate	Experimental	Partial elements (1)	Steady-state temperatures (1)
Numerical simulation of bolted steel connections in fire using explicit dynamic analysis	Yu, Hongxia, et al. (2008)	Bolted	Modelling	Partial elements (1)	Transient linear temperatures (1)
Experimental investigation of the behaviour of fin plate connections in fire	Yu, Hongxia, et al. (2009)	Fin plate	Experimental	Partial elements (1)	Steady-state temperatures (1)
Modelling of bolted angle connections in fire	Daryan, Amir Saedi; Yahyai, Mahmood (2009)	Bolted angle (web and flange)	Modelling	Partial elements (1)	Transient linear temperatures (1)
Fire response of steel semi-rigid beam-column moment connections	Mao, C.J., et al. (2009)	Semi-rigid moment	Experimental/ Modelling	Sub-frame assembly (3)	Steady-state (1) & Standard fire (2)
Modeling and Behaviour of Steel Plate Connections Subject to Various Fire Scenarios	Garlock, Maria E.; Selamet, Serdar (2010)	Shear single plate	Modelling	Sub-frame assembly (3)	Parametric fires (4)

The safety of common steel beam/column connections in fire	Wang, Y.C., et al. (2010)	Extended endplate, fin plate, flexible endplate, web cleat, and flush endplate	Experimental	Partial elements (1) & Sub-frame assemblies (3)	Steady-state (1) & Standard fire (2)
The study of welded semi-rigid connections in fire	Pakala, Purushotham, et al. (2012)	Shear double angle	Experimental	Sub-frame assembly (3)	Parametric fires (4)
Fire resistance of steel shear connections	Selamet, Serdar; Garlock, Maria E. (2014)	Shear double angle	Modelling	Partial elements (1)	Parametric fires (4)
Progressive failure modelling and ductility demand of steel beam-to-column connections in fire	Sun, Ruirui, et al. (2015)	Bolted	Modelling	Partial elements (1)	Standard fire (2)
Numerical evaluation of the effects of fire on steel connections Part 1: Simulation techniques	Rahnavard, Rohola; Thomas, Robert J. (2018)	Bolted	Modelling	Partial elements (1)	Transient linear temperatures (1)
Time-dependent response of flush endplate connections to fire temperatures	Morovat, Mohammed Ali, et al. (2018)	Flush endplate	Modelling	Partial elements (1)	Steady-state temperatures (1)
Behaviour and Limit States of Long-Span Composite Floor Beams with Simple Shear Connections Subject to Compartment Fires: Experimental Evaluation	Choe, Lisa, et al. (2020)	Shear	Experimental	Sub-frame assembly (3)	Transient linear and Steady-state temperatures (1)
Strategies to increase the survivability of steel connections in fire	Safari, Pouria; Broujerdian, Vahid (2020)	Fin plate, flexible endplate, flush endplate, web cleat, and extended endplate	Modelling	Partial elements (1)	Standard fire (2)

4.3 Experimental Analysis of Steel Beam-to-Column Connections Exposed to Fire

4.3.1 Building Case Study

The structural design of the case study considered is based upon a relatively new building in London, UK known as the Scalpel. This building, shown in Figure 4.1, is a 38-storey skyscraper with a unique layout designed in accordance with the Eurocode (UK) building standards. True to its name, the structure has many angled facades terminating in a sharp point similar to a medical scalpel. This uncommon geometry resulted in the building having inclined columns both leaning inwards and outwards. Where the inclined columns join the vertical column, the inclined forces need to be properly designed.

Figure 4. 1: The Scalpel in London, UK with its angled facades.
Photograph captured by thesis supervisor.

The fire design of the Scalpel was performed by ARUP London and was one of the early buildings to use the Travelling Fire Methodology, described in Section 2.2.3. The Scalpel has become recognized for its fire design and its unusual floor plans and has created a precedent in the

UK for using the Travelling Fire Methodology. Through a collaboration with ARUP UK, one beam-to-column setup was identified to be used within this analysis, shown in Figure 4.2 below.

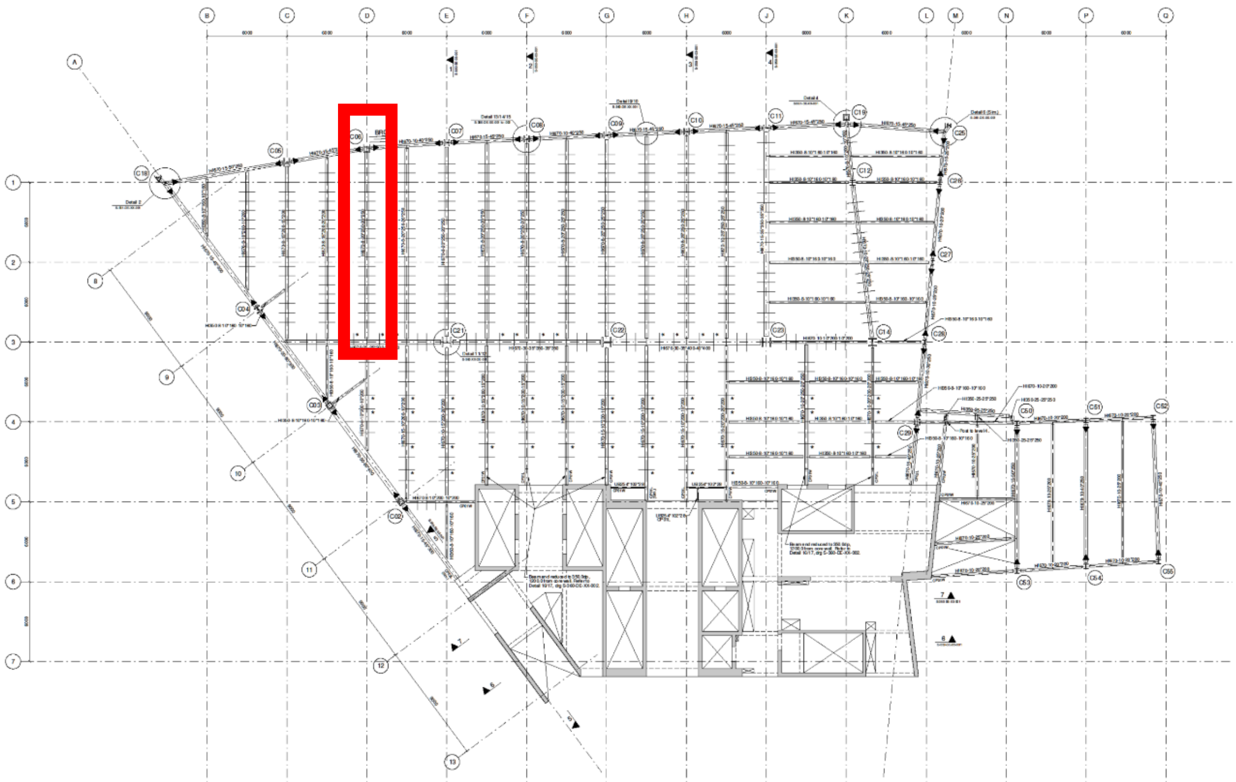

Figure 4. 2: Floor plan of the scalpel, with the area used for the analysis highlighted in red (preliminary plans provided by ARUP UK, used with permission and not representative of the final design).

The identified beam-to-column setup had original dimensions much larger than the York University High Bay Lab could test and therefore required defensible scaling. The selected beam was a PG670-10-25*200-30*200 common steel and the selected column was a HK450-65-65*450-65*450-0. These are equivalent to a W-section with depth of 670mm, width of 200mm, web thickness of 10mm and flange thicknesses of 25mm & 30mm (top and bottom flange respectively), and a square built-up HSS-section with width of 450mm and thickness of 65mm, respectively. For these experiments to be defensible, the scaled setup needed to behave in the same manner as would the original. This meant the scaling was not performed based on the section dimensions. Instead, the sections were scaled while keeping the relative rotational stiffness constant. The rotational stiffness of fixed members can be calculated as:

$$k = \frac{4EI}{L} \qquad \text{(Eq. 4.1)}$$

where

k = rotational stiffness (N mm)

E = elastic modulus (MPa)

I = moment of inertia (mm^4)

L = length (mm)

The relative rotational stiffness in this setup was defined as the ratio of the stiffness of the beam to the stiffness of the column. To fit within the High Bay strong floor dimensions, the original set up was scaled down by a factor of four. The scaling calculations have been included in Appendix E. Based upon this scaling, the chosen beam was a W410x85 and the chosen column was an HSS406x406x16.

The floorplan shown in Figure 4.2 and steel dimensions were provided by ARUP UK based upon a preliminary design of the Scalpel and are not representative of the final as-built structure. In order to be used as part of this experimental study, some aspects of the design were also modified to address specific needs and limitations. The case study simply provides a baseline of a real, built structure and allows for the investigation of similar physical responses that could be expected within a similar building.

4.3.2 Experimental Setup

Using a scaled version of the design of the Scalpel which conserves a representative connection stiffness, a series of three fire experiments of various thermal exposure durations were undertaken to produce a conceptual framework for more exhaustive testing that would eventually lead to a computational tool kit. The focus within these initial experiments was to identify the thermal distribution into the connections when a steel beam was subjected to a localized fire at its center, as well as the deformations observed due to thermal expansion within the beam and connections.

The experimental setup is shown within Figure 4.3 and includes two one-storey linear vertical hollow square columns with a W-section connected between them. These have been specified, as mentioned above as HSS406x406x16 and W410x85, respectively. The columns are 1500mm tall, with the connection centreline located at 1090mm. The beam is 1934 mm long, specified to fit between the two columns which needed to be tied at specific locations. The columns are fixed upon a base plate which allowed for the setup to be tied into the strong floor in the High Bay Lab in fixed condition as shown in Figure 4.4.

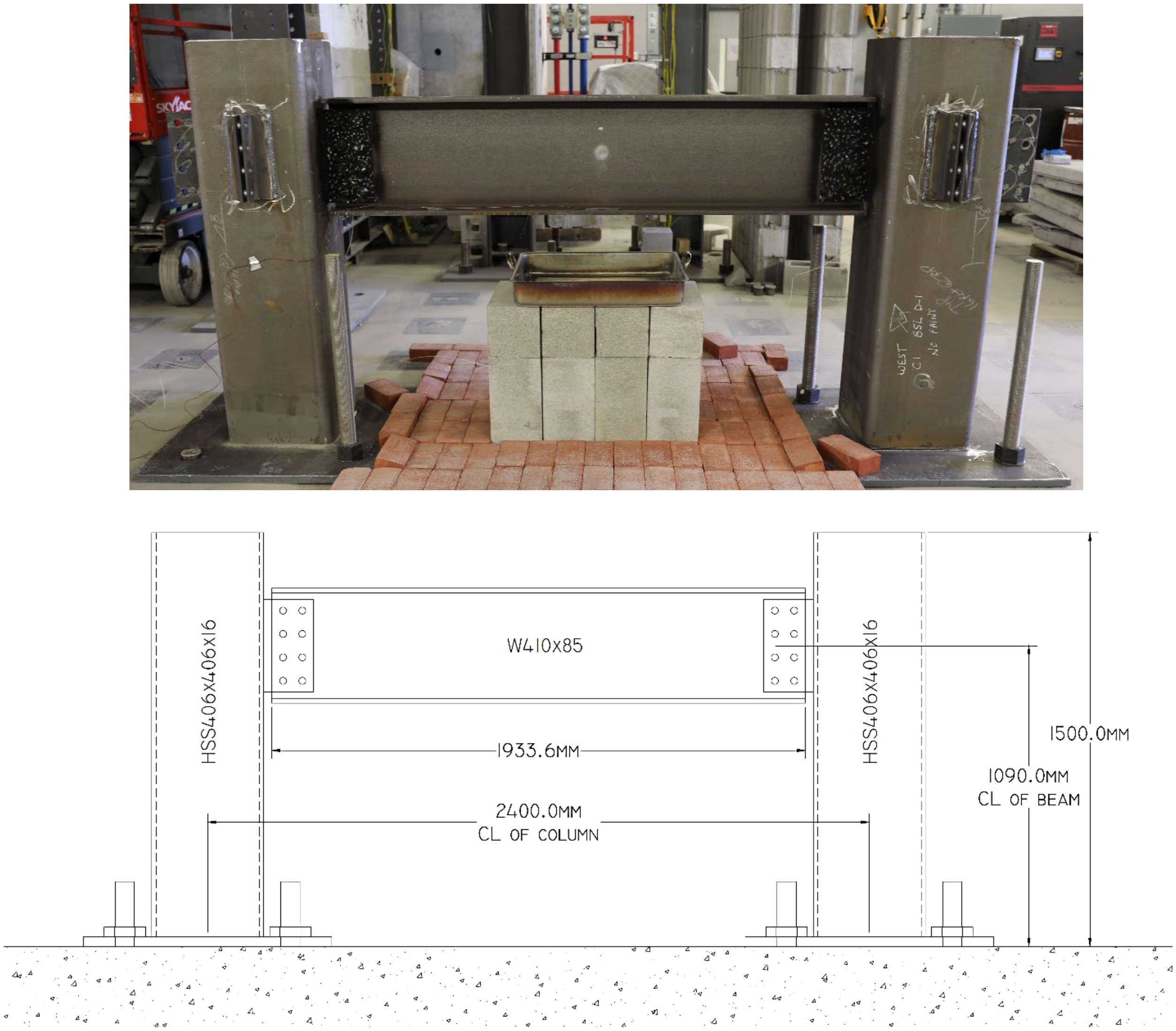

Figure 4. 3: Beam-to-column connection experimental setup.

The connections between the beam and columns were designed by Benson Steel Ltd, a manufacturing organization for the Canadian steel industry located near Toronto which provide their clients with connection design, allowing the welding and detailing to typical construction. This ensured that the connections would be designed as is currently done in Canada, as ultimately although the building was found in Europe guidance, it is intended for North American standards as well[1]. Benson Steel was allowed to design four different connection types with varying ductility, of which three were specified simply as being shear tab, single angle and double angle. As part of the herein experiments, only the shear tab will be used, however future experiments discussed in Section 4.5 will examine the behaviour of the remaining connection types. The shear tab connection is the least ductile of the specified connections and considered brittle. The varying ductility between connections will allow for the observation of differing behaviour under thermal exposure.

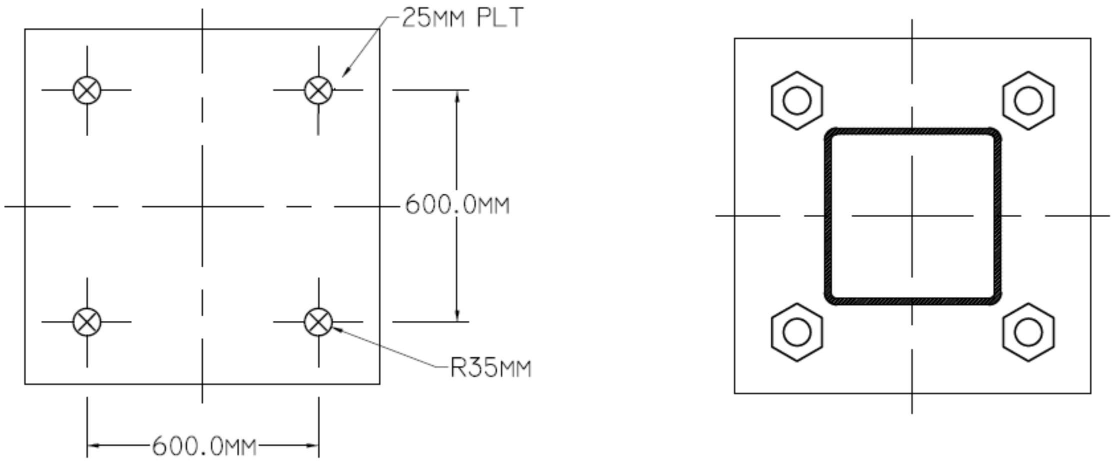

Figure 4. 4: Base plate for the columns.

The connection detailing and calculations for the shear tab were provided and verified, and are included in Appendix F. These were reviewed to ensure they meet the requirements of this study, after which Benson Steel procured and produced the specified sections. Since the beam-to-column setup was scaled using relative stiffness, the connection forces could not be taken from the design of the Scalpel. The scaling ratio from the relative stiffness may not be applicable to the forces directly. Instead, the connection forces were found based on 95% of the capacity of the beam member. The connection forces were calculated using ambient conditions, unlike the connections in the Scalpel which would have increased capacities following a detailed structural fire analysis. This was done since it is representative of design performed in Canada, where a prescriptive design is used, and the connection fire capacities are not considered. Since the

[1] The thesis author could not consider an as-built construction in North America as the industrial partner's presence was European in nature. The fact that the Scalpel was built to European performance-based design extends the usefulness of the research to have more international feedback. Canadian design differs typically in construction structures such as Bay-Adelaide in Toronto are built using truss style composite floors which would not be common internationally as preferences in design differ.

connection specified is a pin-connection, it was designed for an applied shear force of 499 kN with a maximum eccentricity of 302 mm.

The deformations were recorded using Digital Image Correlation, and narrow spectral illumination was used on the images captured at the centre span to filter out the flames from the photographs. This method is described within [152], [153] and allows visual observation of strains and deflections within the flame area, such as the lower flange at the centre of the span. Narrow spectrum illumination is a technology developed by NIST and refined at York University where high intensity blue LEDs illuminate a target and selective color filters are used to filter light bands above blue. This then allows see through fire measurement [153]. Three Canon EOS 5Ds (Mark IV) 30.4 MPx cameras were used to capture the movements within the experimental setup. All cameras were synced to take photos at five second intervals starting at the same time as the data acquisition system for the thermocouples started recording. Previous testing, as highlighted above, has shown this interval to be satisfactory and consistent in recording. The connections, which were far enough from the pool fire, were painted black with white speckles to provide a high-contrast pattern to facilitate DIC analysis. The DIC analysis for multiple locations along the experimental setup was performed used the GeoPIV RG software [154]. The procedure for measuring strains and deformations is achieved through post-processing where pre-recorded imagery is compared and contrasted using the software to discern minute changes that can then be calculated manually as strain or deflections. Following best practices as defined in Gales *et al*. [155], error is minimized. This DIC technique has been shown to be accurate for monitoring displacements in [156], [157]. This thesis' scope is not to improve upon this technique though researchers are actively improving the technique and minimizing error.

Six K-type thermocouples were used to record the steel temperatures at set points within the experimental setup. Each thermocouple was attached to the steel using ceramic fibre wool and aluminium tape padding, as demonstrated in Figure 4.5. This is to shield the thermocouple from influences of surrounding heat and allow it to record the actual temperature of the steel. The location of each of the thermocouples has been illustrated in Figure 4.6, below. These locations allowed for the understanding of the thermal gradients that would occur throughout the depth and span of the beam during thermal exposures.

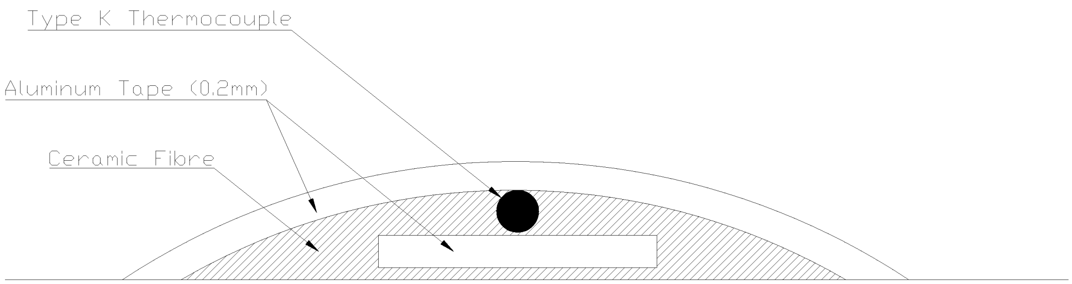

Figure 4. 5: Thermocouple treatment.

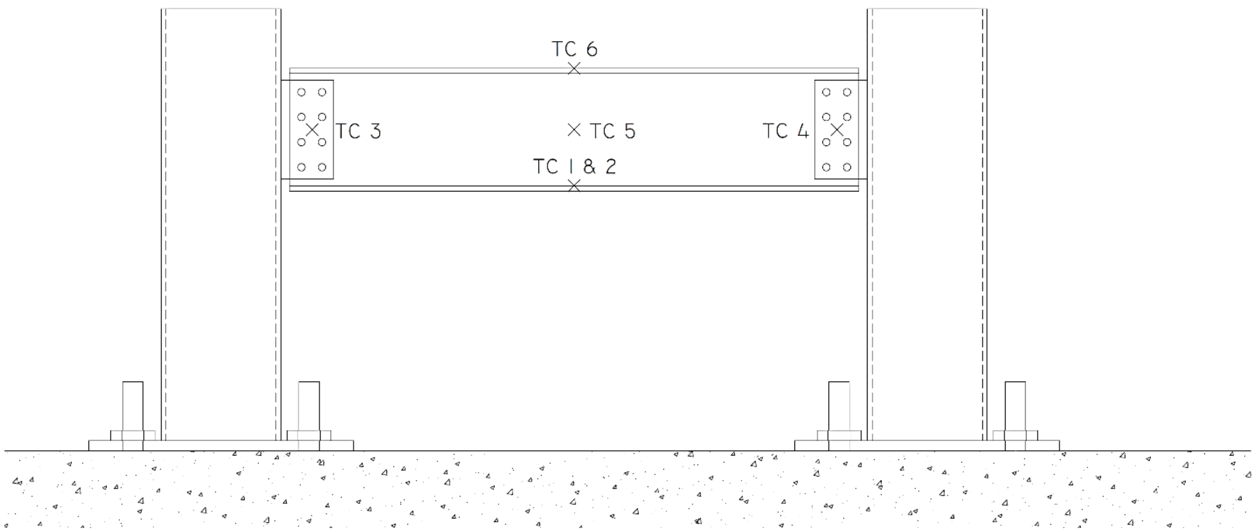

Figure 4. 6: Thermocouple location.

4.3.3 Thermal Exposures

Three different thermal exposures were used to identify the behaviour within the experimental setup. These included a short, a medium and a long methanol pool fire which created temperatures in excess of 700ºC. Each exposure was then allowed to cool, naturally, until all steel temperatures were measured to be below 100ºC. Due to the soot production of other fuels such as acetone or kerosene, it was decided to use methanol, which produces the least soot. Using a fuel like acetone, which generates higher temperatures, would not permit as effective use of the narrow spectrum illumination, and would sacrifice the ability of recording the movement of the beam. The research by Chorlton *et al*. [156] describes the process used to determine the most suitable fuel type and volume used in these experiments.

For each fire exposure, the specified amount of fuel was placed in a 0.48 m x 0.6 m pan, located 200 mm below the centre of the beam, to create the desired fire exposure. This created an incident heat of the order of 700°C to 800°C, as characterized within [158] and shown in Figure 4.7. For the short, medium and long experiments, 4 L, 8 L and 16 L were used. This resulted in overall exposure times of 12 minutes, 20 minutes, and 38 minutes respectively. As each experiment reached different steel temperatures, they required different cooling periods, which were recorded as 39 minutes, 47 minutes, and 106 minutes respectively.

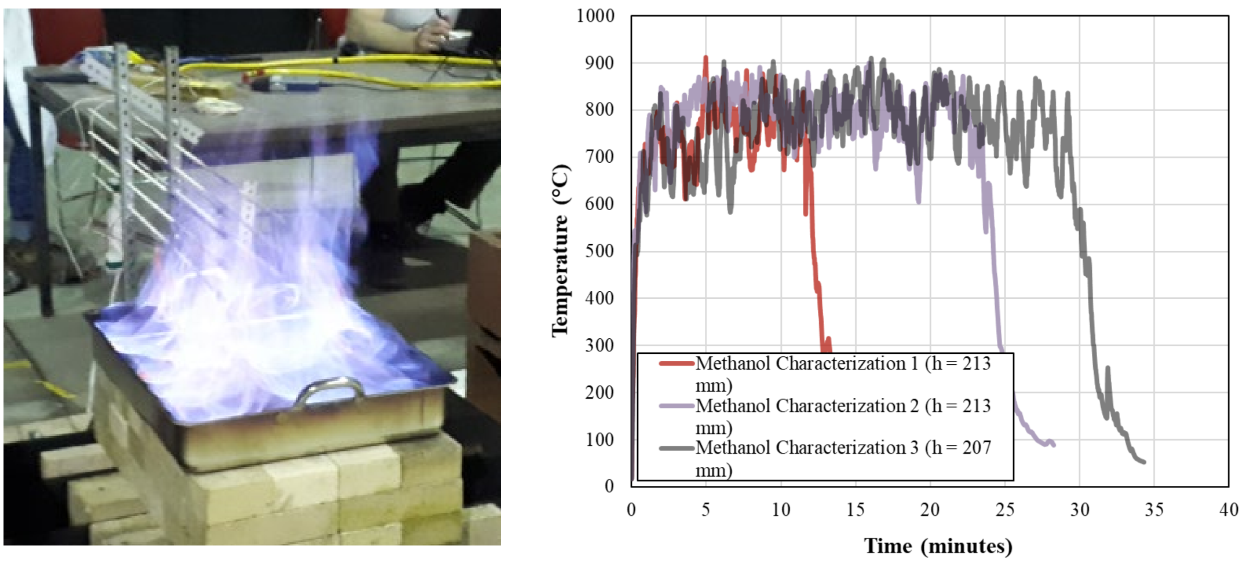

Figure 4. 7: Characterization of methanol pool fire, reproduced with permission from [158].

As mentioned above, it was important to consider consistent crudeness when planning these experiments. This meant selecting a form of localized heating that would resemble as close as possible to a real fire where the connectors may not be exposed to fire. This was the reason a pool fire was selected instead of a radiant heater, where a set radiation would have been chosen. A pool fire inflicts both radiative and convective heating, which is more similar to a real fire. It also allows for heating on multiple faces of the beam, as a real fire would, instead of only having heat directed at the sides facing the heater. This creates a more realistic heat distribution within the steel beam. It should be acknowledged however that a pool fire is not a real exposure and does not follow the curve of a standard fire. It does, however, allow for controlled observational study of the structural system under exposure that can be repeated. Although a furnace test could have also been used based on the consistent crudeness principle, it would have removed the ability to use digital image correlation and narrow spectrum illumination. The ability of a pool fire to create a repeatable thermal exposure while allowing for deformations to be tracked during the experiment was key. This type of exposure, combined with narrow spectrum illumination, was previously used by Nicoletta *et al*. [159] and Chorlton *et al*. [156]. These studies used similar setups consisting of methanol pool fires and narrow spectrum illumination to monitor material strains and behaviours, and this technique has allowed for repeatable results in creating a representative fire exposure of near 700ºC.

There are some limitations to the chosen thermal exposures that need to be considered prior to the presentation of the analysis and results of the experiments. Due to time constraints, only one experiment was performed for each fire length. Further experiments are needed to validate the results found within this study. The experimental program presented herein is preliminary and conceptual in that it is designed to identify areas in which a full test series may be built, which is beyond the scope of the current thesis.

4.3.4 Results

4.3.4.1 Temperature Distribution

As shown in Figure 4.6, the thermocouples were placed along the beam and connections to obtain an understanding of the thermal distribution that would occur due to a localized fire. Figure 4.8 plots the temperatures recorded with the thermocouples. Some experimental errors occurred during testing, including a thermocouple detaching (TC5 in the long experiment after 80 minutes) and improper wiring (TC1 in the short experiment). The improper wiring in the short experiment was identified as TC1 and TC2 should be following similar temperature increases as both are located on the bottom flange at the centre of the beam (behaviour shown in the medium and long tests). The wiring was therefore corrected before undertaking the following experiments. As for the detaching thermocouple, this thermocouple recorded the air temperature for approximately 30 minutes before it was noticed, after which it was reattached to the web of the beam. The air temperature segment of the data was omitted from the graph; however, it is assumed that the web would have cooled in a similar fashion to the previous experiments, following the decrease in temperature observed throughout the beam. Some flaming on the aluminium tape was observed around 10 minutes for the medium and long experiments, however, this does not seem to have affected the temperature readings as no fluctuations in temperature occurred as a result. This is likely due to the insulation being provided on top of the thermocouple.

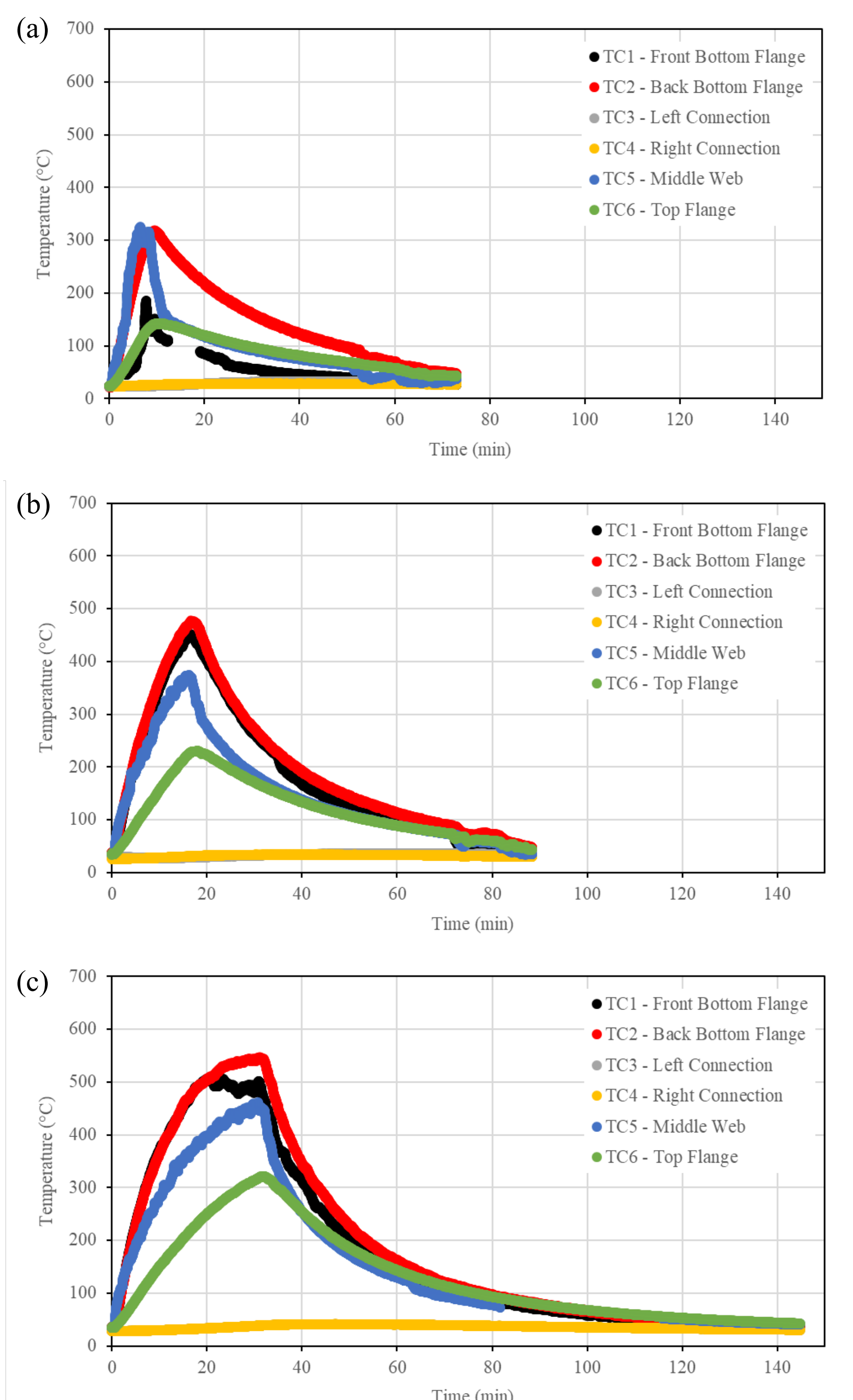

Figure 4. 8: Temperature distribution through experimental setup for (a) short, (b) medium, and (c) long thermal exposures.

From Figure 4.8, it can be observed that the temperature distribution within the beam behaves as expected. The pool fire would have reached temperatures of around 700°C, as demonstrated by previous pool fire studies using the same configuration [156], [157]. This temperature is illustrated as the upper bounds of the figures and indicates the degree of heat transfer observed. The bottom flange and web above the pool fire saw rapid increases in temperatures as the pool fire reached steady state, however the beam never reached the fire temperature, although this might have been possible with a longer exposure. The top flange heats up at a slower rate, as it was outside the reach of the flames, relying on heat transfer through the web and the hot air convection to heat up.

The connections located approximately 950 mm away from the centre of the pool fire, experienced minimal temperature increases. The maximum connection temperatures recorded were as 33°C, 37°C and 43°C for the short, medium and long experiments respectively. The peaks in the connection temperatures occurred after the fire was already extinguished, illustrating the delay in the heating of the connections due heat transfer through the beam. The low connection temperatures are related to the experimental setup and should not be utilized as proof of low connection temperatures within real fire scenarios. The experimental setup applied a short thermal exposure to the centre of the beam, which does not capture all possible scenarios a structure may experience.

While the heating times were recorded as 12 minutes, 20 minutes, and 38 minutes, it can be seen that cooling within the longer duration tests (medium and long) started slightly before the end of the fire. This has to do with the nature of a pool fire and the shape of the pan used to hold the methanol, shown in Figure 4.9. When there is limited fuel in the pan, the fuel tends to gather within the corners of this rectangular pan, therefore not impacting the centre of the beam with flames. Future testing using pool fires should instead utilise a modified W section of appropriate volume and dimension as shown within Figure 4.9.

Figure 4. 9: Pool fire fuel pan used for the current study (left) with lower ridge around edge where fuel pools, compared to suggested *non-flexible* fuel pan (right) that ensures even distribution.

Rapid cooling is observed immediately after the thermal exposure, with the hot steel rapidly losing heat to the surrounding cool air. The cooling rate reduces as the temperature difference decreases. Within the short and medium graphs, some abnormal temperature rates are observed which can be attributed to the water misting applied to aid in cooling. The misting was not applied on the long test to prevent any temperature disruption. The temperatures measured are what would be expected from heating from one side and indicate that the connectors themselves are not fire exposed taking only thermal loading from the beam itself as is intended in this experimental procedure. No changes to the experimental methodology are necessary with respect to heating, other than the reconfiguration of the heating pan.

The temperatures recorded for the lower flanges of the experimental setup can be compared to simple hand calculations described in BS EN 1993-1-2 clause 4.2.5 [160]. This step-by-step method assumes that all the heat entering the section is used to raise its temperature, however it does assume there is an equivalent uniform temperature distribution in the cross-section. The experimental setup does not have uniform temperature distribution, however, this calculation is only used for comparison. The step-by-step method requires the steel temperature to be calculated for set time intervals, which is calculated using the following equation:

$$\Delta T_s = k_{sh} \frac{F}{V} \frac{1}{\rho_s c_s} \left[h_c \left(T_f - T_s \right) + \sigma \varepsilon \left(T_f^4 - T_s^4 \right) \right] \Delta t \qquad \textit{(Eq. 4.2)}$$

where

ΔT_s = the change in steel temperature (°C)

k_{sh} = the correction factor for the shadow effect (assumed 1.0 to be conservative)

$\frac{F}{V}$ = the section factor (m^{-1}), where F can be taken as the exposed surface area (m^2) and V the volume of the beam (m^3)

ρ_s = the density, or unit mass, of the steel (kg/m^3)

h_c = the convective heat transfer coefficient, taken as 25 W/m^2 K

T_f = the temperature of the fire (°C)

T_s = the current steel temperature (°C)

σ = the Stefan-Boltzmann constant, 5.67×10^{-8} $W/m^2\ K^4$

ε = the resultant emissivity, given as 0.8 in BS EN 1991-1-2

Δt = the time interval (s), limited to 5 seconds

c_s = the specific heat of the steel, which changes with the steel temperature using the following equation:

$$c_s = 425 + 0.773T_s - 1.69E^{-3}T_s^2 + 2.22E^{-6}T_s^3 \qquad \text{when } 20°C \leq T_s \leq 600°C$$

$$c_s = 666 + \frac{13002}{738 - T_s} \qquad \text{when } 600°C < T_s \leq 735°C$$

For the comparison of the hand calculations to the recorded temperatures, some assumptions were made. The section factor was calculated using only the bottom flange, assuming all exposed surfaces were uniformly heated (perimeter of flange minus the thickness of the web). This resulted in a section factor of 177.63 m^{-1}. The density of the steel was specified as 7850 kg/m^3. The temperature of the fire was held constant at 700°C, which conservatively follows the pool fire characterization previously discussed. The time interval was assumed to be 5 seconds for this analysis. Using Equation 4.2, Figure 4.10 was generated which illustrated the predicted steel temperatures based on this simple hand calculations. It can be seen that the hand-calculations follow the steel temperature increase for the first 10 minutes of the experiments, after which the calculations become conservative by over-estimating the temperatures. It should be noted that Equation 4.2 was developed with the assumption of a standard fire heating regime instead of a localised fire. This may explain some of the discrepancies between the calculated and recorded temperatures as there would have been heat transfer to the surrounding beam within the experimental setup, which would not have occurred in the calculations as the entire section would have heated up uniformly. Additional comparisons could be undertaken by developing a more complex model of the experimental setup, however, this was outside the scope of this thesis.

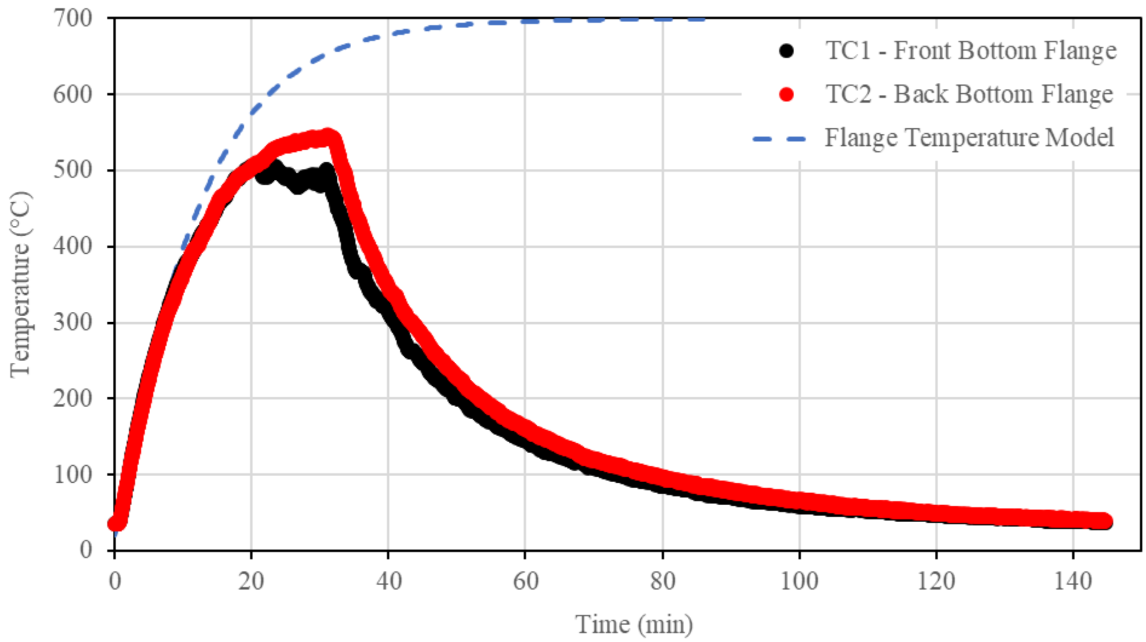

Figure 4. 10: A hand-calculation model of the bottom flange temperature compared against the recorded bottom flange temperatures from the long thermal exposure experiment.

4.3.4.2 Centre Span Deflection

The deflections at the centre span of the beam were recorded using Digital Image Correlation (DIC) with narrowband illumination technologies. Using the GeoPIV RG software [154], points were selected at the upper edge of the top flange and lower edge of the bottom flange located along the centreline of the beam. No paint was applied within this area due to the uncertainty of the behaviour of the paint at such high temperatures – potentially releasing toxic fumes that may not be fully extracted by the exhaust system without harming researchers. In future tests, different paint configurations designed for high temperature testing may be considered. These could include the use of engine motor paints; however, they would require a degree of validation as they have not been used by researchers to date. Traditional deflection measurement were not used as there was still a degree of substantial heat above the beam which could have influenced the connecting cords. This would cause the deflection meters themselves to thermally expand and give false measurements. As shown in Figure 4.11, the contrast between the open space behind the beam (black within the figure) and the edges of the flanges were used instead. Errors also occurred within the DIC tracking at the centre span due to the heat radiation distorting the image. This haze induced some data static, resulting in the need to splice the data. This was done using the methodology described and validated within [158].

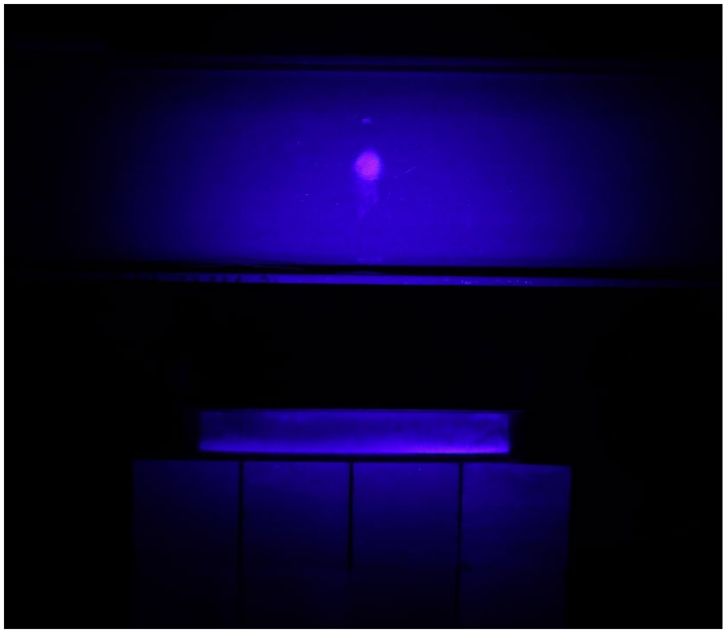

Figure 4. 11: Photograph captured for DIC analysis at the centre span, with the flame filtered out using narrow spectrum illumination.

The recorded deflections are shown within Figure 4.12 for all three tests. It should be noted that the time, shown on the x-axis, is different for the graphs of each test. The deflection, y-axis, however, was kept the same to allow comparison between experiments. The deflections of both the top and bottom flanges of the beam were recorded to capture the thermal bowing that would occur, as well as the thermal expansion that would occur predominantly within the bottom flange. This can be seen to occur within all three tests, with a difference of approximately 1.5 mm occurring.

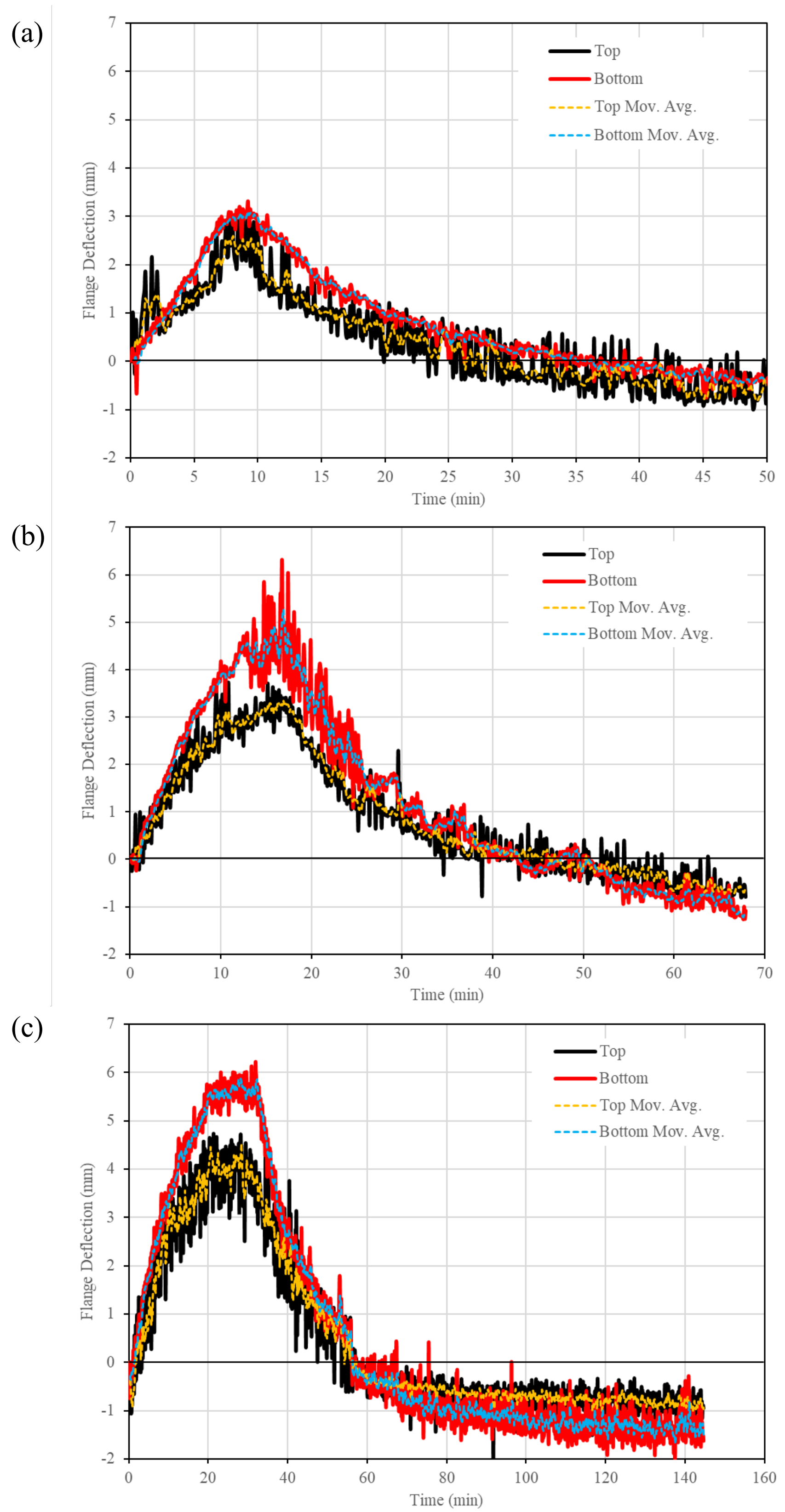

Figure 4. 12: Deflections at the centre span for (a) short, (b) medium, and (c) long thermal exposures.

The maximum deflections were seen to occur shortly before the pool fire was extinguished but around the peak temperatures experience within the beam, due to the same reasoning explained previously regarding the end of the pool fire. These were recorded as shown within Table 4.2 for each test. As Figure 4.12 illustrates, there was some noise in the data, previously explained as the haze distorting the image. The values included within Table 4.2 are therefore calculated using a rolling average, with a period of 5, to remove over estimations of the deflection. Figure 4.12, also, illustrates that an upward camber is created within the beam as it cools down. This upward camber is minimal, remaining below 2mm at the centre of the span for all tests. This type of behaviour occurs within the experiments as the beam was unloaded. The maximum upward camber for each test is also included in Table 4.2.

Table 4. 2: Averaged maximum deflection and upward cambers for each thermal exposure and flange.

Thermal Exposure	Flange	Maximum Deflection (mm)	Maximum Upward Camber (mm)
Short	Top	2.6	0.8
	Bottom	3.1	0.5
Medium	Top	3.3	0.7
	Bottom	5.3	1.2
Long	Top	4.5	1.0
	Bottom	5.9	1.7

Additional research will be needed to undertake studies regarding the sensitives of heat haze. It is suggested to use a grid-based system behind an open fire to calibrate the degree of haze so as to quantify the uncertainty seen in the tests. This endeavour would need careful experimentation as out of plane effects would need to be minimized by placing the grid within the flame itself and preventing the grid form excessive thermal straining itself. Nevertheless, based upon the measurements observed, clear trends in deflection match the expected trends as would be predicted for an object heating and then subsequently cooling.

4.3.4.3 Restraining Forces

The forces applied onto the connections due to the thermal expansion and contraction of the beam were calculated by using Digital Image Correlation to track the movement of the column. For the purpose of force calculations, the columns within the experimental setup can be assumed to behave similarly to a cantilevered beam with a point load applied at the height of the connections. This provides a conceptual model of the forces being generated, however the real behaviour of the frame would have the loads at equilibrium. The columns were fixed to the High

Bay floor using tie rods, preventing displacements or rotations. Using DIC to measure the outward deflection of the columns, the force applied by the beam onto the connection can be calculated using the following equation:

$$\Delta = \frac{PL^3}{3EI} \qquad \textit{(Eq. 4.3)}$$

where

Δ = deflection (mm)

P = force (N)

L = length (mm)

E = elastic modulus (MPa)

I = moment of inertia (mm^4)

The column was an HSS406x406x16 section made of 350W steel, therefore the elastic modulus was 200,000MPa and the moment of inertia for the section was 606 x10^6 mm^4. This equation can then be rearranged to find the force applied, *P,* as follows:

$$P = \frac{3EI\Delta}{L^3} \qquad \textit{(Eq. 4.4)}$$

The deflections of the columns were measured using connections affixed onto the column facing the camera. This allowed for the deflections to be measured as close to the centreline of the column as possible. The deflections were measured at the top and bottom of these additional connections. This is because it is known that the force applied from the beam into the connection would not be centred into the connection. The lengths used for the calculations were 1236 mm and 944 mm, for the top and bottom deflections respectively. The thermal expansion would be focused within the bottom flange of the beam, causing an uneven distribution of stress within the beam. The neutral axis for the forces within the connection would therefore be lower than the centre of the connection. Measuring the deflection of the column at two points therefore allows to capture the forces within the top and bottom of the connection and will be used to determine the centreline of the force within the connections. This behaviour of higher forces within the bottom of the connection can be seen within Figure 4.13.

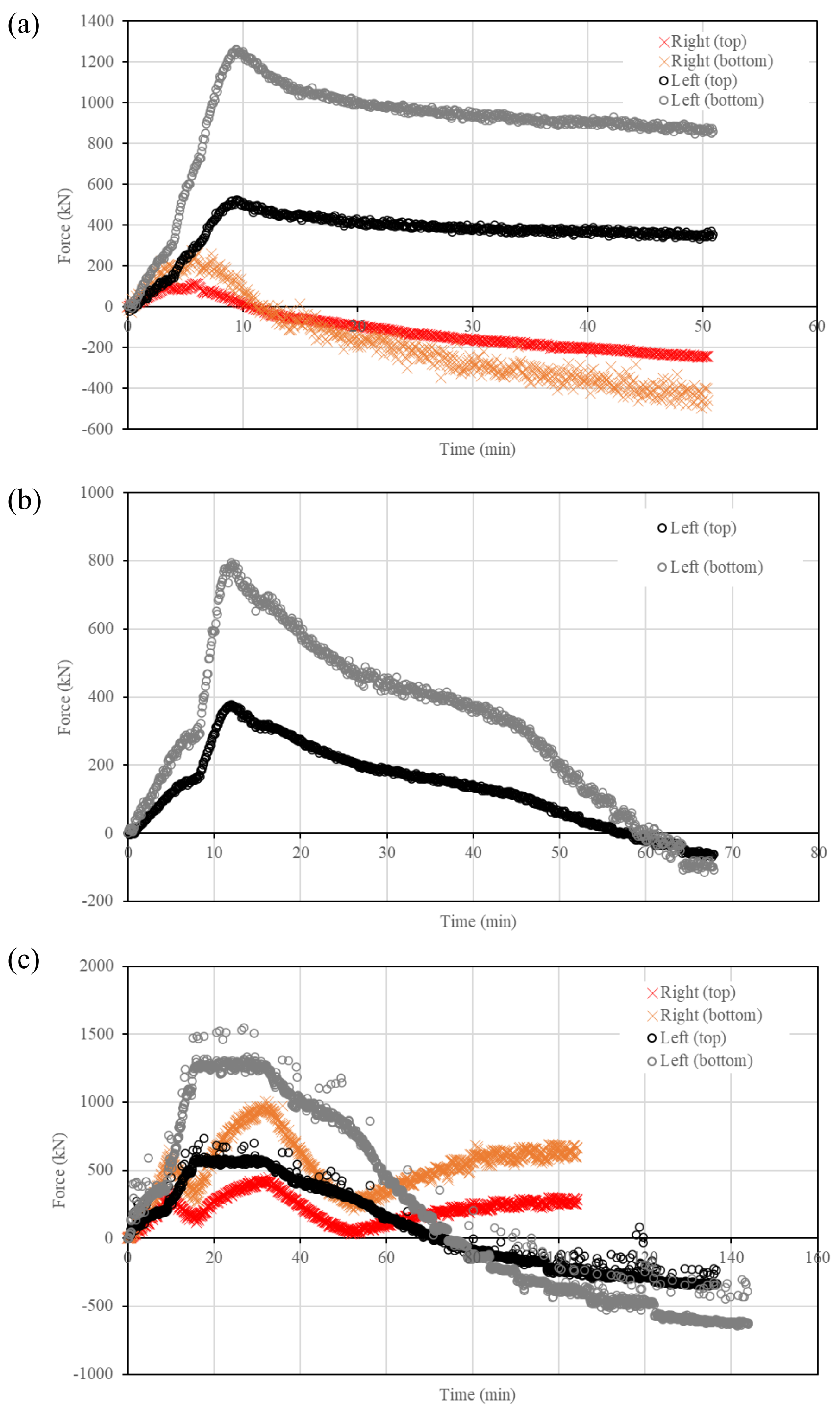

Figure 4. 13: Restraining forces generated by the thermal expansion and contraction of the beam for (a) short, (b) medium, and (c) long thermal exposures.

It can also be seen that the forces distribute unevenly between the columns. This can be caused by various factors within the experimental setup. The thermal exposure could have been slightly un-uniform, causing more heating closer to one column, or simply the fuel pan was not perfectly centred. As well, it is possible that the connections were unevenly tightened, allowing for more friction to occur within one connection as compared to the other. It was also noted that the High Bay floor was uneven when placing the column, as well as the base plates of the columns showed some bowing due to the heat generated from welding the column onto the plate. These would have allowed for some minor movement of the columns.

The camera capturing the right connection during the long thermal exposure failed after approximately 105 minutes, creating a gap in the data after this point. As well, the forces experienced within the right column during the medium length experiment were omitted from the graph due to inaccuracies in the recording. Enough data was generated however to provide an approximate maximum restraining force applied. This, with those experienced within all columns for all tests, have been gathered within Table 4.3. The value of the force applied the neutral axis within the columns would be resolved as a value between both the top and bottom force. The total of the resolved forces being applied to both columns is then the force being generated due to the thermal expansion and contraction of the beam.

Table 4. 3: Maximum restraining forces generated into both connections for each thermal exposure

Thermal Exposure	**Connection**	**Measurement**	**Maximum Restraining Force (kN)**
Short	Left	Top	525
		Bottom	1263
	Right	Top	116
		Bottom	276
Medium	Left	Top	380
		Bottom	794
	Right	Top	596
		Bottom	1397
Long	Left	Top	733
		Bottom	1545
	Right	Top	446
		Bottom	1005

The observed forces as measured are simplified theoretical hand calculations which assume the columns themselves do not move and have full rigidity in their connection to the strong floor for which is not a guaranteed assumption. This method of calculation examines each column as an individual system, which differs from the loads experienced within the frame. In future research, the column should be tied to the floor non-destructively and pushed utilising an actuator with known load, similarly to the method used by Gales [161]. This will then allow a load to deflection correlation of the columns to be established, which will subsequently allow a re-analysis of the data presented herein. Access to the strong wall to undertake this type of testing was not available for the thesis timeline but is necessary for future validation of the test procedure and accurate quantification of the forces observed in the thesis. Using the current methodology, the forces observed are within the range of what would be expected in a fire as numerically predicted within Smith and Gales [80].

4.3.5 Discussion

The experiments described herein were the first experiments to assume realistic rigidity or restraint within the experimental setup. The majority of the limited experiments in literature, discussed within Section 4.2, examined other behaviours experienced within connections without considering the restraint of a real structure. The experiments described herein were based upon a real structure and, while they were scaled down, this was done in a manner to ensure the rigidity and restraint that would be demonstrated within the case study structure would remain between the frame elements.

The recorded thermocouple data demonstrates the repeatability of a methanol pool fire. From Figure 4.8, it can be seen that the steel within the flaming region (on the lower web) consistently followed the same increase between all three tests. This demonstrates the consistent thermal exposure generated by a methanol pool fire, even when different volumes of fuel are used. The recorded temperatures were also compared to a hand-calculation model, described within the Eurocodes for steel temperatures during uniform thermal exposures. The comparison illustrated the hand calculations are conservative compared to the experimental results, however, this may be due to the localised heating used.

The deflection measurements demonstrated that unloaded beams exposed to temperatures below 700°C for short durations experience no permanent thermal degradation. Instead, a slight upward camber is observed due to the uneven cooling that occurs within the assembly. This type of upward camber would not be expected within a structure after a fire, leading to possible inspections coming to incorrect conclusion that no damage was incurred. Had the beam been loaded, which would be more likely within a structure, there would have been creep (plastic) deformations that would have occurred in addition to this upward camber, resulting in the camber not being observable. It is therefore not possible for a typical structure to have a final upward deflection after fire, and this camber is a result of the experimental setup.

The restraining forces recorded within these experiments were quantified to be within the range of 1.5 MN when combined between both connections. These are within the range of what is currently used within current design, with previous literature indicating forces within the range of 1 MN for a steel frame building exposed to standard, parametric and travelling fires [80]. This is a preliminary step to validate that forces currently used within design are representative of forces that could possibly be generated within a steel frame structure. There has been limited experimental studies to date that validated the large connection forces experienced due to thermal exposure [140], [162], [163]. To date, the scale of the forces being generated into connections has largely relied on theoretical knowledge.

4.4 Development of Alternative Solutions

The overall objective of this chapter was to produce preliminary guidance for safer and more resilient steel connection design, which would eventually be provided within Annex K provisions in CSA S16-19: Design of Steel Structures. These provisions would allow for practitioners to undertake alternative solutions that consider the overall behaviour of the structure instead of designing each member for certain capacity requirements. The current version of Annex K was developed following its American counterpart, ANSI/AISC 360 Appendix 4. While there are advantages to the interchangeability between Canadian and American design, allowing for practitioners to design projects within both jurisdictions, it does result in dismission of the advancements seen in European design. The performance-based approach based on agreed upon design objectives undertaken in Europe is not compatible with the prescriptive based solution seen in Canada. Simply following the American design limits the use of the innovations and research progress observed within different jurisdictions. This includes the possibility of innovations captured within the Canadian research field, where design techniques are considering timber on steel composite structures. Due to its development based upon its American counterpart, Annex K is not built to consider the hybrid innovations required within the Canadian design context.

This thesis is the preliminary work required to create a foundation study in which a true fundamental understanding of steel frame construction can be understood and provide made-in-Canada approaches for design. While the experiments are, in all respects, conceptual and preliminary, they do justify the following updates towards providing rational design in Canadian industry. The following changes are suggested [2]:

- **K.2.2.6 Active fire protection systems**
  - <u>*The presence of active fire protection systems may be accounted for in the determination of the fuel load density.*</u>
  - (Delete all the original wording in this section)
- **K.2.5.3.2 Design by simple methods of analysis**
  - *The methods of analysis in this Clause may be used for the evaluation of the performance of individual members <u>and components</u> at elevated temperatures during exposure to fire.*
  - *<u>Restraint stiffness</u> applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.*
  - <u>*Provisions in this section do not preclude those contained in Section K.2.5.1.*</u>

The rationale for these changes is justified based on the experimental evidence presented within this chapter, as follows:

- **For K.2.2.6 provisional changes**: The original clause does not address the actual fuel which will decide the applicable design fire for design. The new text clarifies that it is essential to understand the actual fuel being used which moves the use of alternative solutions within the standard closer to acceptable solutions such as a travelling fire in large compartments or any other design fire which is a function of fuel. The previous clause is not clear to what design fire can be used and has allowance for crude design fires that are not a function of fuel load. The tests herein show that the resulting connection forces are a function of the applied heat on the beam from the fuel so therefore accurate quantification of fuel load is necessary for

[2] The thesis author was assisted with this proposal of changes through discussion with Kevin LaMalva of Warrington Fire and her supervisor who chairs the Working Group 10 of CSA S16 for Annex K. The suggested changes will be presented for discussion and revision for the next CSA S16 Annex K edition in 2025.

designing connectors. This change also ensures the design is properly undertaken and the structure would be able to resist a realistic fire should the sprinkler system, or other active systems, not be properly engaged.

- **For K.2.5.3.2 provisional changes**: These changes address the fact that connectors need to be considered in design, and that the overall stiffness of the structure needs to be accounted for. It also implies that the general structural integrity needs to be considered as the restraining frame determines the restraint of the beam and the connector forces accordingly.

The changes proposed above are unintrusive to the existing built infrastructure as they do not preclude the designs that were undertaken prior to the changes coming into effect. They do, however, move Annex K towards clarity and begin the process towards performance-based procedures.

It is still necessary that alternative solution development be explored and published to allow Canadian standards to move towards international usability for all design frameworks. Additional changes should be considered to Annex K once the herein study, including the expanded research procedure, has come to completion. There should also be a continued dialogue with ANSI/AISC 360 committees to ensure interchangeability between the two jurisdictions.

4.5 Limitations and Gaps in Literature

The results obtained as part of this study are only the preliminary steps required to develop alternative solutions for steel connections. Additional research is required, which will require refinement and additional experiments. Modelling technologies will also be required to develop a thorough understanding of steel connections when exposed to thermal exposures. The experiments described above highlighted areas that required additional research and validation. These included but are not limited to:

- characterizing the effect of the heat haze on the DIC analysis;
- experiments with the pool fire located near a connection to understand the effect of thermal distribution and force generation in a non-uniformly heated assembly; and
- additional experiments following a modified procedure to ensure repeatability within the results observed.

The herein study has allowed for the characterization of the expected forces caused by non-standard fires. The restraining forces recorded within the experiment discussed herein, as well as the additional experiments undertaken for repeatability, will be used to validate a nonlinear Finite Element Analysis (FEA) model. The induced movement found within the FEA models will be applied to the beam-to-column setups using an actuator to determine if the connections can withstand the forces induced by thermal loading. The experiments will test the various of connections discussed in Section 4.3.2. A novel hybrid testing method, under different realistic thermal exposures, similar to the thermal exposure selection chosen in [164] could also be explored without having to heat the setups. The hybrid test could simulate fire induced forces on the connection as seen in experiments. The numerical model would be used to calculate the thermal effects the assembly would experience over time and these movements would be programmed into the actuator. This novel hybrid experimental methodology may be a simpler and cheaper method than performing full scale fire experiments and would create more accessibility for industry to use alternative solutions.

4.6 Summary

This study has produced a preliminary understanding of forces being generated into the steel beam-to-column connections when a localized thermal exposure is applied to the centre span of the beam. A methanol pool fire was used, following the methodology validated within previous studies [156], [159], and was shown to be a repeatable form of thermal exposure by the temperatures recorded during the experiments. The magnitude of the restraining forces recorded within the experiments has seen limited validation within literature, relying mostly on theoretical knowledge. The realistic rigidity and restraint captured within the experimental setup begins to develop foundational knowledge of the behaviour of steel beam-to-column assemblies when exposed to fire. Preliminary recommendations for the development of the relevant Canadian design code, Annex K within CSA S16-19, have been outlined.

Practitioners are in need of tools which facilitate the improvement of connection design in order to create resilient infrastructure that can resist thermal loading. When a realistic experimental design is used, it allows for accurate forces and behaviours to be generated. This generates greater understanding of this area which requires additional research. The experiments discussed within this chapter are a preliminary and conceptual research program, which are needed to generate a framework for further testing which will lead to analytical tools and further guidance regarding steel connections in fire.

The results of the study establish the first step towards updating Annex K for the design of steel beam-to-column connections. The experiments were undertaken based on the Scalpel as a case study building and were exposed to three different fire lengths. The results illustrated repeatability for using a methanol pool fire as a non-standard thermal exposure, with steel temperatures being repeatable between all three experiments. The recorded beam deflections were within the range of 2.6 to 5.9 mm, with an upward camber captured at the end of every test. This upward camber is not representative of the fires within real structures due to the beam being unloaded during the thermal exposure. The restraining forces recorded during the experiments were calculated assuming fixed conditions to the High Bay strong floor. The forces were within the magnitudes of 1.5 MN, which is comparable to values used in design and obtained during numerical modeling [80]. The results of the experiments were used to recommend unintrusive changes to Annex K, allowing Canadian design to move towards clarity and additional performance-based procedures.

The movement towards alternative solutions and performance-based design in Canada requires more robust codes that include innovations and research progress illustrative of Canadian and international design. The research herein has demonstrated the need to further study the behaviour of steel beam-to-column connections, to ensure safe and resilient infrastructure if exposed to fire.

Chapter 5:
Conclusions and Recommendations

5.1 Summary

Fire Safety Engineering is an important aspect of safe and resilient infrastructure design. With many jurisdictions internationally having transitioned to performance-based design, Canadian structural fire design is currently restricted by its reliance upon predominantly prescriptive based approaches. Within the National Building Code of Canada (NBCC) [1], there is the flexibility to utilize alternative solutions and more advanced computational practices to optimize the fire protection design. The concept of fire resilience and performance-based fire design is relatively novel in Canada, and there is a need to provide the required information to consultants to be able to achieve it. It should be recognized that Canada is in a position that allows the guidance to move towards a more performance-based fire design approach. Alternative solutions and performance-based designs are not necessarily applicable in non-developed countries, highlighting the need for additional research into such jurisdictions. Preliminary work taken to understand how fire is addressed in non-developed countries is discussed within the journal paper provided in Appendix G. The research undertaken as part of this thesis is the first step in the development of generalized frameworks from which researchers, modellers and practitioners can create alternative solutions for Canadian jurisdictions.

A review of the structure fire design methodologies used in Canada and internationally was provided in Chapter 2 to contextualize the research in subsequent chapters. The development of prescriptive fire design and its evolution over time within Canada is discussed. The relatively recent change of the National Building Code of Canada to objective-based is compared to the prior movement of international jurisdictions to performance-based, demonstrating the lessons learned and reliance of Canada on its American counterparts. A general understanding of structural fire design in Canada for concrete and steel structures is provided, allowing for the identification of research gaps when compared to the European performance-based design which incorporate more innovation and research. The flexibility for Canadian practitioners to undertake alternative solutions has been limited by the lack of knowledge and resources of the Authorities Having Jurisdiction to evaluate and approve such designs. There is a need for simple analytical tools and additional knowledge which can enable rational fire design.

To address the need for knowledge to undertake alternative solutions, Chapter 3 describes the methodology to follow to develop an acceptance criterion for the fire design of unbonded post-tensioned (UPT) concrete slabs. This demonstrates some first steps needed for the development of alternative solution frameworks. Unbonded post-tensioned (UPT) concrete is one of many commonly used structural systems that required additional guidance for fire design. The effect of different design fires on a UPT concrete case study structure is analyzed and discussed. The analysis examined two possible types of prestressing steel, identifying at which length and temperature the prestressing steel would meet failure criteria. For the first time for UPT structures, a comparison of the effect of uniform and travelling fires was reported. The stress relaxation model developed by Gales *et al*. [2] was validated and used to establish preliminary definitions of critical design thermal boundaries for UPT concrete flat-plate slabs. The analysis identified tendon rupture failures at elevated temperatures for prestressing steel with specific composition and quality. Different steel composition however showed no vulnerabilities to strength failure (tendon rupture) but illustrated severe stress relaxation potential. This indicates that the quality of prestressing steel is imperative to consider in design. The analysis also illustrated the need to consider travelling and localised design fires due to the vulnerability of unbonded tendons to localized tendons. Structural systems such as UPT concrete are of high complexity and require additional development of analytical tools to undertake alternative solutions. It was demonstrated that critical thermal boundaries would allow for simpler and safer structural fire design.

Technology and methodology generation is also important for the research and development of alternative solutions. The fire performance of steel beam-to-column connections was considered experimentally in Chapter 4. Identified through accidental fires and experimental programs as the most vulnerable component of steel structures, the beam-to-column connections are often designed for only vertical load transfer. In a fire scenario, the thermal expansion and contraction of beams inflict large restraining forces horizontally on the connections. An experimental program of three pool fire experiments of various thermal exposures was undertaken to produce preliminary validations of structural fire modelling. The realistic rigidity and stiffness of the experimental setup allowed for accurate forces and behaviours to be generated, expanding the understanding of the forces and deformations that occur in steel beam-to-column connections. The thermal distribution, centre span deflections and column rotations were recorded during the experiments, using thermocouples and Digital Image Correlation combined with narrow spectrum illumination. These experiments were preliminary and conceptual in nature, allowing for the development of a future hybrid experimental framework, which was outside the scope of this thesis. This research program was the first step that will lead to the generation of analytical tools and further guidance regarding steel connections in fire. The results of the study provided preliminary guidance towards updating Annex K within CSA S16-19.

5.2 Conclusions

The following key conclusions were drawn on the basis of the review, computational modelling and experimental testing regimes described in this thesis:

- There exists the possibility within the Canadian design codes to consider more alternative solutions, yet there is a lack of education and resources available to the practitioners and Authorities Having Jurisdiction to undertake and approve such designs;
- The first steps towards the definition of critical design thermal boundaries for unbonded post-tensioned (UPT) concrete flat-plate slabs were undertaken, identifying a maximum tendon length of 42 m for steels that exhibit similar behavioural characteristics as Steel II within similar sized compartments as the case study considered;
- Within the UPT concrete case study structure, the slow travelling fire (5%) was identified as the most critical design fire, demonstrating the importance of considering a range of design fires as the localised heating effect and longer duration of slow travelling fires can create more onerous conditions within structures;
- The steel beam-to-column connection experimental program produced a preliminary understanding of forces being generated into connections when a localized exposure is applied to the centre span of the beam for a structure with realistic rigidity and stiffness, with restraining forces recorded at magnitudes of 1.5 MN, validating the forces seen through structural modelling discussed in literature;
- The steel beam-to-column connection experiments are a preliminary and conceptual research program, which are needed to generate a framework for further testing which will lead to analytical tools and further guidance regarding steel connections in fire; and
- Preliminary recommendations for the development of the steel in fire Canadian design code, Annex K within CSA S16-19, were outlined.

Several additional experimental-based conclusions can be drawn from the research presented herein, including that:

- The quality of the prestressing steel is imperative to consider in the design as it influences the strength and stress relaxation profiles;
- The use of a methanol pool fire as a non-standard thermal exposure was shown to be repeatable; and
- The deflections recorded for the steel beam-to-column connection experiments were within the range of 2.6 to 5.9 mm, with an upward camber captured at the end of every experiment due to the cooling phase and experimental setup that did not include structural loading on the beam.

5.3 Design Recommendations

Overall, the research presented herein expands the knowledge of the fire performance of selected structural systems. This is a first step to increase the confidence of engineers designing structures and the Authorities Having Jurisdiction who approve the designs. Alternative solutions in structural fire design have the ability to provide greater architectural freedom, ensuring that critical infrastructure is resilient and capable of resisting thermal loading. The research presented herein are the preliminary steps required to develop alternative solutions for Canadian design. The following design recommendations are considered conservative to update existing guidance.

Defining Acceptance Criterion for Unbonded Post-Tensioned Concrete Structures Exposed to Fire

Practitioners can utilise the methodology described within Chapter 3 to establish a set of temperature contours for the design of specific structures, including considerations for appropriately defined failure criteria with the stakeholders. The methodology utilises the validated tendon relaxation model to generate contour plots showing the temperature at which varying prestressing tendon lengths experience a stress (50 % stress relaxation) or strength (where the stress surpasses the strength of the tendon) failure. Practitioners can use the generated contour plots to identify whether their designed tendon length meets the design criterion, i.e. would not fail due to stress or strength failure due to fire.

Recommended Modifications for Clauses in CSA S16-19 Design of Steel Structures' Annex K

Preliminary recommendations for the next CSA S16 Annex K were developed in Chapter 4 based upon the results of the experimental program. These unintrusive changes help to move the current steel fire design, outlined within Annex K, towards clarity and begin the process of moving steel fire design towards performance-based procedures. The recommended changes are as follows:

- **K.2.2.6 Active fire protection systems**
    - *The presence of active fire protection systems may be accounted for in the determination of the fuel load density.*
    - (Delete all the original wording in this section)
- **K.2.5.3.2 Design by simple methods of analysis**
    - *The methods of analysis in this Clause may be used for the evaluation of the performance of individual members and components at elevated temperatures during exposure to fire.*
    - *Restraint stiffness applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.*
    - *Provisions in this section do not preclude those contained in Section K.2.5.1.*

These changes help to clarify the existing clauses by highlighting the need to consider the actual fuel load and subsequently realistic design fires, as well as the response of the entire building instead of single elements. These recommendations can help achieve a higher standard in Canadian design, allowing for more international interchangeability. The changes are conceptually based, helping to validate European results in a Canadian framework. They will allow Canadian design to generate innovation and research developed locally and internationally, with the potential of eventually influencing the development of other standards.

5.4 Research Recommendations

The following is a list of research recommendations from Chapter 3 which should be considered:

- **Bonded post-tension concrete tendon stress relaxation model** – The tendon stress relaxation model used within Chapter 3 has only been validated for unbonded prestressing steel, limiting the results and acceptance criterion developed to unbonded post-tensioned concrete design. Additional experiments are needed to develop and validate a model for bonded prestressing steel to allow the methodology described in Chapter 3 to be used.

- **Additional tendon modelling** – Further modelling is required for different configurations and scenarios to fully develop an acceptance criterion due to the highly complex behaviour of prestressing steel. This includes fires travelling in different directions, as well as considering different types and profiles of tendons.
- **Additional experiments for different types of prestressing steel** – The results discussed in Chapter 3 identified that the composition and quality of the prestressing steel influences its stress and strength behaviour at elevated temperatures. Further studies considering a broader range of prestressing steel could help identify what composition would perform best under thermal loading in order to ensure more resilient designs.
- **Thermal-mechanical relations of load induced thermal strains (LITS)** – Further experimental studies are required to help our understanding of the effects of LITS on UPT flat-plate concrete slabs exposed to various fires, and to improve and permit the modelling capabilities of this behaviour. This is required it a generalized design acceptance criterion is to be developed.
- **Fire dynamics of large compartments** – Further studies are necessary to fully characterize the fire dynamics that occur within large compartments. This would help with structural computational modelling as it would allow for a more accurate modelling of the thermal boundary and subsequent heat transfer. These efforts are currently underway by various researchers.
- **Spalling experiments** – Prestressing steel tendons are highly susceptible to damage when exposed to high temperatures. When spalling occurs, there is a possibility that the tendons would be exposed directly to the fire. Despite considerable efforts, there is still a lack of understanding regarding the phenomenon of fire-induced spalling of concrete cover. Future research is required to understand this phenomenon and develop methods that can limit and remove the occurrence of spalling.

The following is a list of research recommendations from Chapter 4 which should be considered:

- **Develop novel hybrid testing methodology for experiments presented** – The experiments undertaken in Chapter 4 were preliminary and conceptual in nature to allow for the generation of a framework for further testing, which will lead to analytical tools and further guidance regarding steel connections in fire. A novel hybrid testing method, utilising forces calculated from Finite Element Analysis, would allow for experiments that consider various different realistic thermal exposure without requiring the setups to be heated. Such a testing methodology would be less costly, in time and money, and would allow for a greater range of variable to be examined.

- **Characterizing the effect of heat haze on the Digital Image Correlation analysis** – Understanding the effect of heat haze on the analysis results is integral for future pool fire experiments using DIC and narrow spectrum illumination. The current methodology requires splicing the data when the haze creates static, however, the uncertainty of this method is unknown. An experimental program that focuses on calibrating the degree of haze to quantify the uncertainty seen in the experiments is needed, and a proposed methodology is outline within Chapter 4 that considers the out of plane effects.
- **Further investigation into the effect of thermal distribution** – Further studies into the effect of thermal distribution into the generation of the forces into the connections from thermal expansion and contraction of beams are necessary to help design for the various possible fire scenarios a structure may experience. This can be undertaken by following a similar experimental methodology as described within Chapter 4 and placing the pool fire at various locations along the beam.
- **Additional experiments following a modified experimental methodology** – Future experiments must be conducted following the modified and improved methodology for the beam-to-column connection experiments, discussed within Chapter 4, to ensure repeatability of the recorded results. Additional experiments could also consider different fire sizes and severity.

Overall, this thesis investigated how alternative solutions for common structural systems can be developed, allowing for future Canadian designs to incorporate alternative solutions for more resilient, safer and economical structures.

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APPENDICES

Appendix A:
The Historical Narrative of the Standard Time and Temperature Heating Curve

From:

Gales, J., Chorlton, B., and Jeanneret, C. (2021) The Historical Narrative of the Standard Time and Temperature Heating Curve. Fire Technology. 57, 529-558.

DOI: 10.1007/s10694-020-01040-7

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ABSTRACT

This review aims to provide additional context to the historical narrative of the development ofthe standard temperature-time heating curve used for the determination of the fire resistanceof structural elements. While historical narratives of the development of the standard time- temperature heating curve exist, there are portions of the timeline with missing contributionsand contributions deserving of additional examination. Herein, additional newly available contributions (owing to recent digitization efforts) from the original standard development cycle not distinctly covered by existing historical narratives are introduced and reviewed. Though some engineers have long been recognized for their contributions to the curve's development, lesser-recognized influences are re-examined. These include contributions to fire resistance testing from Sylvanus Reed, that are acknowledged for the first time in a contemporary light. Practitioners will find discussion from the temperature-time heating curve's development period that is useful for current philosophical discussions pertaining to the curve's use for combustible material testing. This study identifies that no currently available historical literature can support the definition of the temperature points which describe the standard time-temperature heating curve. This reinforces contemporary discussion that the heating curve lacks scientific basis in its representation of a real fire.

Keywords: Fire drills, Aging Populations, Dependent behaviour, Evacuation, Behavioural data,Movement time

1. INTRODUCTION

The standard fire test is a common and globally applied fire resistance metric. Its advantage lies in its simplicity, convenience in repeatability, and the fact that it has been used for more than a century. It therefore has a wealth of experience for testing performance of structures and building products. This allows the construction industry to move at a fast pace based on precedent. Historically however, questions have been posed by practitioners and researchers on the applicability of the standard temperature-time heating furnace test as a single qualification metric for structural fire resistance design purposes. Questions were primarily derived from the unrealistic nature of the standard temperature-time curve as a fire and the potential limitationsto this [1, 2]. For example, not considering cooling phases have been shown to be detrimental forcertain structural configurations such as connections [3] or for passive fire protection products such as gypsum boards [4]. Other researchers have argued that non-uniform fires may indeed lead to a different and more onerous structural responses. In intumescent paint application to steel structures, slower heating regimes have been shown sometimes to be more detrimental asthe paint may not always activate [5]. All these limitations need to be considered by standardization committees when developing guidance specific to structural design and/ or complementary testing of fire protection systems.

More recently, the questions about the use of the standard fire resistance framework has been directed towards its application to timber members [6, 7]. This is due to timber's combustible nature which provides additional heating fuel in the furnace, which consequently affects the actual applied fuel needed to control the time-temperature curve (see Figure 1). The subsequentconsiderations have led to recent debates on how the fire resistance of timber and even concrete structural elements can be compared using the same standard when fundamentally, the thermalinput (or boundary condition) is different between incombustible to combustible materials. Therefore, this above discussion raises a renewed interest in the historical basis of the frameworkof the standard temperature-time heating curve that permits its use on timber elements. Becausethe standard temperature-time heating curve has not been changed significantly since its conceptual definition in 1916, it is important to understand what data it was created upon, and if the founders envisioned this modern boundary condition paradigm in standardization. That knowledge can help determine the standard temperature-time heating curve's applicability to current designs and in identifying if an evolution could be necessary for standardized testing in the future.

Figure 1. Standard Fire Resistance Test of a Wooden Floor (at test conclusion - author's photos).

Herein, the authors attempt to complement the historical narrative of the standard temperature-time heating curve that is used within the current fire resistance structural element test. Withinthe current ASTM standard [8], a brief historical narrative is provided in its commentaries and the reader is directed to an original historical review of fire resistance testing that was presentedin the Fire Technology journal over forty years ago [9, 10]. That two-part paper represents a historical narrative of standard fire resistance testing by Babrauskas and Williamson, titled: *The Historical Basis of Fire Resistance Testing* [9, 10]. These papers referenced in the standard examined the multitude of tests upon which it is assumed the standard temperature-time heating curve is based upon. The Babrauskas and Williamson study reviewed tests undertaken between the 1880s and 1918 which show the development of standardized fire testing of floors,walls, columns and doors. These papers had a focus on those occurring in North America, however, some reference is made to European tests which also had an impact. That research outlined the lack of changes to the time-temperature heating curve, the basic test apparatus, andsome of the testing criteria as it was created in 1916 and conferred upon for standard use in the1917 column test program. Babrauskas and Williamson attribute the development of the standard fire curve as a consensus of stakeholders examining previous tests and developing an appropriate time-temperature envelope. They detail the history of the attempts to create universal exposure and test standard citing the European works of Edwin Sachs and the BFPC (British Fire Prevention Committee) test criterion (discussed herein). It became apparent in NFPAand ASTM meetings that followed that North America intended to modify that test criterion for use, however, the adaptation was rejected as noted in the two-part paper. No commentary is directly given by Babrauskas and Williamson in their paper regarding the BFPC's use of multiple fire intensities to define protective classes, though such an effort is often applied in international structural fire design by considering a structure's response to a family of fires. Herein, we will also show Edwin Sachs' efforts were not the first attempt to consider a range of fires to assess building materials. In their papers, Babrauskas and Williamson focus on providing a historical narrative leaving interpretation largely to the reader. Figure 2 illustrates the historical narrative of key publications that were identified by Babrauskas and Williamson. The narrative is not completely exhaustive and there are portions of the timeline with missing contributions or contributions which have profound influence and are deserving of additional examination.

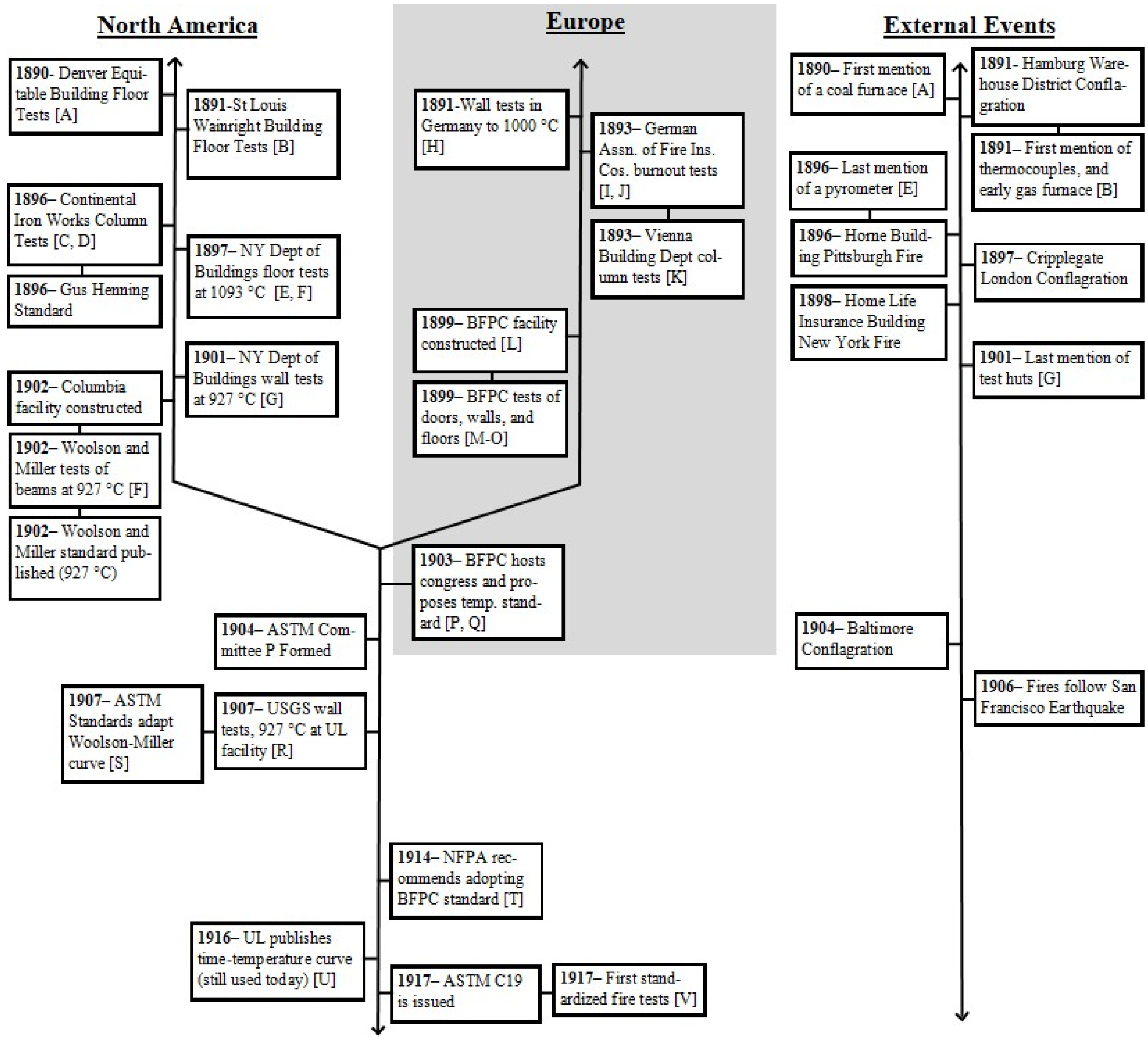

Figure 2a. 1890-1917 Timeline of the Development of the Standard Time and Temperature Curve (as adapted from source material presented in Babrauskas and Williamson [9, 10]).

A. The American Architect and Building News, Vol. 31 (March 28, 1891), pp. 195-201.
B. Engineering Record, Vol. 23 (May 7, 1892), pp. 376-377.
C. Sachs, E. O, "Fire Tests with Unprotected Columns," Red Book No. 11; British Fire Prevention Committee, London, 1899.
D. Engineering News, Vol. 36 (August 6, 1896), pp. 92-94.
E. Engineering Record, Vol. 35 (January 2, 1897), pp. 93-94; (May 29, 1897), pp.558-560; also Vol. 36 (September 18, 1897) pp. 337-340; (September 25, 1897), pp. 359-363; (October 2, 1897), pp. 382-387 and pp. 402-405.
F. Woolson, I. H., and Miller, R. P., "Fire Tests of Floors in the United States," Proceedings, Sixth Congress; International Association for Testing Materials, New York, 1912.
G. Engineering News, Vol. 46 (December 26, 1901), pp. 482-486 and 489-490.
H. Bohme, Mittheilungen, K. Tech. Versuchsanstalt, Berlin, Vol. 9 (1891), pp. 268-270.
I. Deutsche Bauzeitung, Vol. 27 (May 6, 1893), pp. 224-227; (May 13, 1893), pp. 241-242; (May 20, 1893), pp. 246-248.
J. Engineering Record, Vol. 26 (October 7, 1893), p. 300; (October 14, 1893), pp. 317-318.
K. Engineering News, Vol. 32 (September 6, 1894), p. 184.
L. Sachs, E. O., "The V. F. P. C. Testing Station," Red Book No. 13; British Fire Prevention Committee, London, 1899.
M. "Fire Tests with Floors -- A Floor by the Expanded Metal Company," Red Book No. 14; British Fire Prevention Committee, London, 1899.
N. "Fire Tests with Partitions," Red Book, No. 22; British Fire Prevention Committee, London, 1899.
O. "Fire Tests with Doors," Red Book, No. 24; British Fire Prevention Committee, London, 1899.
P. The Official Congress Report, First International Fire Prevention Congress; British Fire Prevention Committee, London, 1903.
Q. Woolson, I. H., Report of Proceedings of the International Fire Prevention Congress; Martin Brown, New York, 1904.
R. Humphrey, R. L., "The Fire-Resistive Properties of Various Building Materials," USGS Bulletin 370; Government Printing Office, Washington, 1909.
S. "Standard Test for Fire-Proof Floor Construction," ASTM Proceedings, Vol. 7 (1907), pp. 179-180. 69
T. "Report of Committee on Fire-Resistive Construction," NFPA Proceedings, Vol. 18 (1914), pp. 216-219.
U. NFPA Quarterly, Vol. 9 (1916), pp. 253-260.
V. Underwriters Laboratories, Fire Tests of Building Columns; Chicago, 1920. Also issued as Bureau of Standards Technical Paper 184.

Figure 2b. 1890-1917 Reference Material for the Development of the Standard Time and Temperature Curve (as adapted from source material presented in Babrauskas and Williamson[9, 10]).

To place that two-part historical narrative paper [9, 10] within context, it is necessary to highlightthe study's origins. This two-part paper mainly derives from Babrauskas' doctoral thesis published in 1976 [11]. Within that thesis the goals of the work are quite clear; that Prof. Brady Williamson believed *"… that fire protection can and should be an engineering discipline, not just a technology guided by traditional roles and ad hoc methods"* and more specifically the work wasto: "… *attempts to examine the major aspects of fire endurance in buildings and provide a self- consistent rationally based framework for design and analysis. Four broad areas of concern are developed. These are the physics of compartment fires, test requirements, design procedures, andhistory of fire endurance requirements and standards. The latter is pivotal for the understandingof the status quo, since it will be shown that the present building code* *provisions are founded largely on studies reported in the 1920' s and earlier - their relationship to the present state of engineering and economics knowledge is not notably strong*."

The goal of our paper's review is to provide further context to the standard temperature-time heating curve's origins and is within similar motivation of those that precede it in their studies over 40 years ago. This communication will examine the now available literary sources that werenot necessarily available (see Section 2) to Babrauskas and Williamson, and the effect of combustible materials to extend the historical narrative. In addition, this paper only considers standard temperature-time heating curve, not other fire resistance test methodologies, procedures and controls reviewed elsewhere.

Herein, it is not attempted to debate merits and pitfalls, nor rationalize consistent crudeness paradigms of fire resistance testing and the introductions of contemporary resilience definitions. The authors' primary aim is to provide practitioners with additional source references that can accurately interpret how this standard temperature-time heating curve framework came to be to provide context for the origins of the curve. We specifically focus on (forgotten) literature fromwithin the time period that the curve was developed. This paper was developed to more thoroughly complete the historical narrative and direct other researchers to where the narrativeis incomplete. The authors leave others with the task of rationally evolving fire tests that fulfill the safety and property protection of various international building and construction codes though this paper may provide useful context to those discussions. Therefore, this study's primary use is to be built upon by others. While it is acknowledged that in some cases subjectivityis required for interpretation of events, the authors have made effort to minimize this where possible and present only factual discussions that have been found to be recorded in literature. The authors therefore encourage the reader to examine the sourced and referenced articles where possible. To the authors' awareness no formal framework defining intent was developed throughout the evolution of the temperature-time heating curve.

Additional contributions from the original development cycle not distinctly covered by existing historical narratives are explored for the first time herein. While in the past some have believed to have found the foundations of the temperature-time heating curve (including that it corresponds to the rate at which wood could be stocked in a fire; and to the melting points of metals) the authors have found no evidence to support these claims. The authors will identify that no literature is (currently) available that supports a basis for the points which describe the standard temperature-time heating curve. The contributions to fire resistance testing from Sylvanus Reed will be acknowledged herein in a contemporary light, particularly his contribution towards integrating real fire dynamics for different building types. Practitioners will find, presented herein, discussion from the time-temperature curve's development period that is useful for current philosophical discussions pertaining to the curve's use for combustible and incombustible structural elements.

2. LITERARY SEARCH METHODOLOGY

There is a significant resource (the previously mentioned two-part study led by Babrauskas and Williamson, then of the University of California) that practitioners utilize when discussing the origins of the standard temperature-time heating curve and likewise the concept of the historical basis of fire resistance testing [9, 10]. By far, those papers are thorough investigations into the subject. Since their publication, however, questions have emerged regarding missing detailing (for instance, in regards to what the temperatures within the curve represent leading some to believe unverified claims). The complete narrative of the curve's evolution is missing. The authorsbelieve that due to the lack of resources available to Babrauskas and Williamson at the time of writing their manuscripts, their narrative could be further expanded. Digitization (as we know today where articles are freely accessible online) did not exist in the 1970s and this can have restricted those authors [9, 10] to provide a complete narrative. Today, a large portion of the source literature cited in [9, 10] are available online and digitized by the University of California for public access. Furthermore, it can be examined that these sources digitized reflect the curriculum of the first fire course that was offered at the University of California (to the authors' knowledge) [1]. It reflects very accurately the time period during which the standard time- temperature heating curve was being developed (1890-1916). The course calendar of that fire course taught at University of California (1914) is provided herein as adapted from the NFPA Quarterly series [12] (see Figure 3).

[1] The Armour Institute of Technology in Chicago, Illinois was the first post-secondary institution in North America tooffer courses in fire protection engineering. First offered in 1903, the fire engineering program ran until the 1980s, surviving the merging of the school with the Lewis Institute to form the Illinois Institute of Technology. The programwas promoted by insurance companies who were seeking specialists in the new methods for fire prevention. [13]

Fireproofing Course, Civil Engineering.

University of California, Department of Civil Engineering.

TESTING LABORATORY
C. DERLETH, Jr., Director. A. C. ALVAREZ, Instructor in Charge.

THE PRINCIPLES AND PRACTICE OF FIRE RESISTIVE STRUCTURAL DESIGN AND FIRE EXTINCTIVE EQUIPMENT.

1. Introduction. Fire Losses in the United States and in Europe Annually during the Last Forty Years.
   - *a.* Summarized Comparative Statistics with Analyses and Conclusions.
   - *b.* Importance of the Personal Factor and the Need of Arousing a Public Opinion to Reduce the Fire Tax.
   - *c.* Fire Hazards.
2. Fire-Resistive Properties of the Materials of Construction. Standard Fire Tests.
   - Natural Building Stones, Clay Brick, Sand-Lime Brick, Terra Cottas, Plasters, Wired Glass, Cast Iron, Wrought Iron, Steel.
3. Corrosion and Preservation of Structural Metal.
   - *a.* The Theories of Corrosion.
   - *b.* Conditions which Accelerate or Inhibit Corrosion.
   - *c.* Protection by Metal Coatings.
     1. Methods of Galvanizing.
     2. Requisites for Zinc Coatings.
     3. Tests of Zinc Coatings.
   - *d.* Protection by Preservative Paints.
     1. Classification of Pigments and Carriers.
     2. Requisites for Paints in Various Exposures.
4. Slow-Burning Construction.
   - *a.* Structural Practice pertaining to Foundations, Walls, Columns, Girders, Floors, Roofs, Opening Coverings.
   - *b.* Adaptation and Advantages.
5. Standard Reinforced Concrete Fire-Resisting Building Construction.
   - Discussion of the Standard recently adopted by the National Fire Protection Association.
6. Protection of Steel Frame Buildings.
   - *a.* Column, Girder and Beam coverings of Concrete, Terra Cottas and Plasters; Their Relative Merits.
   - *b.* Floor, Partition, Ceiling and Roof Construction.
   - *c.* Coverings for Openings.
     Wire glass windows.
     Tin clad, sheet steel and rolling doors and shutters.

Under 6*a*, 6*b* and 6*c* the discussion includes a survey of the floor tests made by the New York Bureau of Buildings under the direction of Mr. Ira

Figure 3a. University of California Fire Course Reading List Part 1 (1914) (from [12]).

H. Woolson, also all the floor and other tests made by the British Fire Prevention Committee. In all cases outlines are given of the present standards as adopted by the National Fire Protection Association.

7. Fire Extinguishing Apparatus.
   - *a.* Sprinkler Systems.
     The Installation and Requisites for:
     1. An Automatic Wet-pipe System.
     2. An Automatic Dry-pipe System.
     3. An Open Pipe System.
   - *b.* Water Supplies.
     Requisites for the following Types:
     1. Gravity Tanks.
     2. Pressure Tanks.
     3. Fire Pumps.
     4. Municipal.
   - *c.* The Requisites and Installation for Standard Standpipe Systems.
   - *d.* Signaling Systems.
     1. Automatic Sprinkler Alarms.
     2. Thermostat Alarms.
     3. Fire Detecting Tube Alarms.
     4. Manual and Supervisory Alarms.
     5. Municipal Fire Alarm Systems.
   - *e.* Chemical Fire Extinguishers.
   - *f.* Discipline of Employees.
   - *g.* Municipal Apparatus.
     1. High pressure systems.
     2. Motor car pumps.
     3. Ladder towers.

8. European Practice.

NOTE.—The text-book read by the students is Freitag's "Fire Prevention and Fire Protection."

As problem work each student designs and draws the plans for a four-story slow-burning mill building equipped with a wet-pipe sprinkler system and standpipe system, together with the necessary storage and means for providing water supply.

References Consulted During the Course.

1. Fire Prevention and Fire Protection. Freitag. John Wiley & Sons, Publishers.
2. Fire Prevention. McKeon.
3. Baltimore Fire. Report of National Fire Protection Association.
4. San Francisco Fire. Reports of:—
   - *a.* United States Geological Survey.
   - *b.* National Fire Protection Association.
   - *c.* American Society of Civil Engineers.
5. Building Construction and Superintendence. Part 2. Kidder.
6. Architects and Builders Handbook. Kidder.
7. Fireproofing of Steel Buildings. Freitag.
8. Reports Insurance Engineering Experiment Station.
9. U. S. G. S. Bulletin 312 on Fire Waste in the U. S.
10. Special Consular Report No. 38 on Fire Waste and Prevention in Foreign Countries.
11. Building Code of the National Board of Fire Underwriters.
12. Rules and Requirements adopted by the National Fire Protection Association.
13. Publications of the British Fire Prevention Committee.
14. Articles on Insurance Engineering, Proceedings of the National Fire Protection Association, Current Engineering Literature.

Figure 3b. University of California Fire Course Reading List Part 2 (1914) (from [12]).

Today, with the novel efforts to improve digitization technology, it is possible to expand upon theabove literary search performed previously by Babrauskas and Williamson [9, 10] by looking at articles beyond the California library collection. With every year, new documents related to the fire engineering research field are digitized by various university and industrial catalogues. Thesenew documents provide a clearer picture on the origin of the standard temperature-time heatingcurve, the standardization test motivation and the philosophy behind its development. Herein, the authors aim to discuss additional period sources that complement previous historical paper compilations on this subject. The authors will restrict the literary search to post-1870 aligning with the various city conflagrations in North America, which are generally accepted as the promptfor the creation of fire resistance philosophies as opposed to fire proofing design. The interestedreader is directed elsewhere for structural fire testing historical information for the period of 1770-1870 when the first documented tests are recorded (see [14-17]), or where relevant within the footnotes of this manuscript are provided.

While parallel efforts to developing standardized fire resistance test procedures for construction elements were mirrored in Europe largely led by the architect Edwin Sachs, the authors will restrict most discussion to North America, and will only discuss the European influences that affected the evolution of the original 1918 ASTM time-temperature heating curve for fireresistance testing.

For this study, an examination of the papers utilized for the Babrauskas and Williamson literaturereview [9, 10] was first conducted. Most of these articles were readily available online and, as aforementioned, they were digitized by the University of California for the *Google Books* and *Hathi Trust* projects. It is of note that these archives also include resources from other universitylibraries thereby expanding the capabilities of a modern literature search on the subject. The combined institutional search of these libraries (online) conducted were provided for examination: the complete works of *NFPA Quarterly*, partial collections of *Engineering News andRecord*, *ASTM committee notes*, American Society of Civil Engineers article database, American Society of Mechanical Engineers article database, American Concrete Institutes database, Canadian Society of Civil Engineers article database, various newspaper archives (*New York Times, New York Tribune, Chicago Tribune* etc.), various pamphlets pertaining to fire proof construction, as well as numerous textbooks on fire protection, and building standards and codes. It should be extended that most professional societies in this time period produced meeting minutes in annual transactions. These were reviewed where available (NFPA, ASCE, ASME, ACI etc.).[2] The resulting literature search considered mainly materials between 1870 and 1927 (1927 as an end date for current copyrights for public posting of digital material at the timethis manuscript was prepared but also correlates to the death of Ira Woolson who instigated the

[2] Several of the figures herein stem from publicly available digitization's from the sources discussed. As such, the quality and resolution of the figures is limited to that of the existing digitization. Many of these documents were digitized as a part of a mass digitization process, for instance through HathiTrust or Google Books. In this process, the document owner will scan the document and send to the organization or loan the document to the organizationfor them to scan. The documents may be scanned using book scanners that feature high quality cameras, with recommendations in place with regards to image quality (though not always strict requirements). Scans may be processed to eliminate noise on the image, and for optical character recognition.

first ASTM fire resistance standard) [3]. While the literature search herein generally considered materials between 1890 and 1927, with supporting materials from post 1980 that elaborate contemporary structural fire design practice, the following decades (1930s to the 1980s) garnered significant research interest and developments into fire dynamics, and eventually in fire modelling [21]. Developments of structural fire design practice and associated fire dynamics beyond the late 1920s should be considered by future work, where existing publications (such as [21],[22], and [23]) may provide a starting point for that research endeavor.

3. THE ORIGINS OF CONTEMPORARY FIRE RESISTANCE

The contemporary definition of "fire resistance" was termed in the late 19th century. This came from an evolution of moving from defining building materials as fire-proof (restriction of combustible materials exposed to high temperature) to fire resistant (how any material performsin a fire – i.e. materials, even incombustible ones, will eventually fail and other metrics of analysisare needed beyond just stability – integrity and insulation for example). The terminology evolved from the aftermath of various city conflagrations. Examination of any major city illustrates theseconflagrations, particularly those seen in Chicago in 1872, and Boston in 1874. The outcome of these city fires led to a surge in ad-hoc (ad-hoc meaning that they were demonstration in test design not necessarily following an established test methodology) fire tests of building elementsthat were often not trusted by the scientific and greater community.

These ad-hoc tests primarily considered new reinforced concrete elements (beams and slabs) which had emerged in the building market in the North East (mostly after the Chicago fire). Between 1870 and 1890, the terminology called "fire-proof" was adopted in practice, where fire-proofing was strictly being defined as incombustible construction [24]. These fire-proof tests were not qualitative in nature and were often performed as a public spectacle. They consisted ofa building element by a material manufacturer (constructed in-situ), supported on stilts. The material would then be "burned" using timber logs in random placement and number. Often, they were unloaded, and measurements (temperature and deflection) during these tests were not recorded. Confidence in the building element was achieved by the non-appearance of a "failure-collapse". These tests had little science to validate the manufacturer's claims, and thereexist little scientific articling or reports that survive today on these tests.

[3] Babrauskas and Williamson note historical papers relating to building materials and fire dating back to the 1700s. Contemporary searches can also show literature dating into the 1600s speaking to building material behavior in fireand fire-fighting technologies. These sources however seem not to speak to standardized fire testing of building materials. Some of the earliest calls for fire testing involve the Barrett-Fox composite floor system in in 1854 in RIBAproceedings [18], which resembles a reinforced composite concrete flooring system. The floor was advertised as being fire-proof, however when presented publicly, criticism highlighted that the new material concrete had questionable performance. In the author's professional experience, when these floors are found in heritagestructures, they are found to be unreliable due to the poor quality of the concrete used and often need to be removed. Those in attendance at these historical meetings stated that the debate could only be resolved with testingof these flooring systems. Thomas Thyatt Lewis (1865) presents a comprehensive overview of Victorian era papers on building materials in fire. In his review, he notes the contributions of James Braidwood to the aspect of materialslosing strength in fire though little experimental evidence was available to quantify the effects [19, 20].

By the 1890s, ad-hoc testing was decreasing. This appears initiated by architects when assigning competing assemblies for design that were being claimed to be fire-proof [25]. This led to the emergence of the concept of fire resistance. Testing began that now considered quantitative performance and attempted to rank competing materials on the basis of a test standard of equivalent thermal assault. Researchers measured temperature (and deflection) in these tests, and the tests of building elements were then compared on these bases. Measurement of deflection of a building element was used to define failure criterion, while collapse after a measurable exposure time was deemed defining failure. One of the first “fire-designed” buildingsusing early principles of fire resistance, the Denver Equitable Building, was constructed in the 1890s. Prior to its construction, the responsible architects were faced with choosing three competing flooring (arch) systems made of terra-cotta, which were said to be fire-proof by their manufacturers [25]. The manufacturers of these competing flooring systems each debated that their products were superior to the other – leaving the architects to resolve this. The architecturalfirm Andrews, Jaques and Rantoul organized a test program to settle the debate where they would rank each flooring system in a comparative coal-stocked fire test. The test utilized the same target temperature of assault for each specimen tested. They specified and recorded gas temperatures. These were measured with platinum wire where temperature was calibrated to electrical resistance. Each platinum wire has its own circuit of copper wire which was connectedto a Wheatstone bridge and galvanometer. The temperatures measured were approximately peaking at 815°C (1500F) with variability (note experimental accuracy of this time period cannotbe relied upon in assessing thermal performance of the materials) and the flooring systems wereranked accordingly to time of collapse. Note that these tests were extensively documented in a test report [25]. They were highlighted briefly in [9, 10], though they were not placed in the context of their significance in attempting to create comparative and standardization principles.Even today, a 17-photo set of loading and failure conditions survives in the Denver Public Library,see Figure 4 for a few sample images illustrating the repeated test procedure with instrumentation.

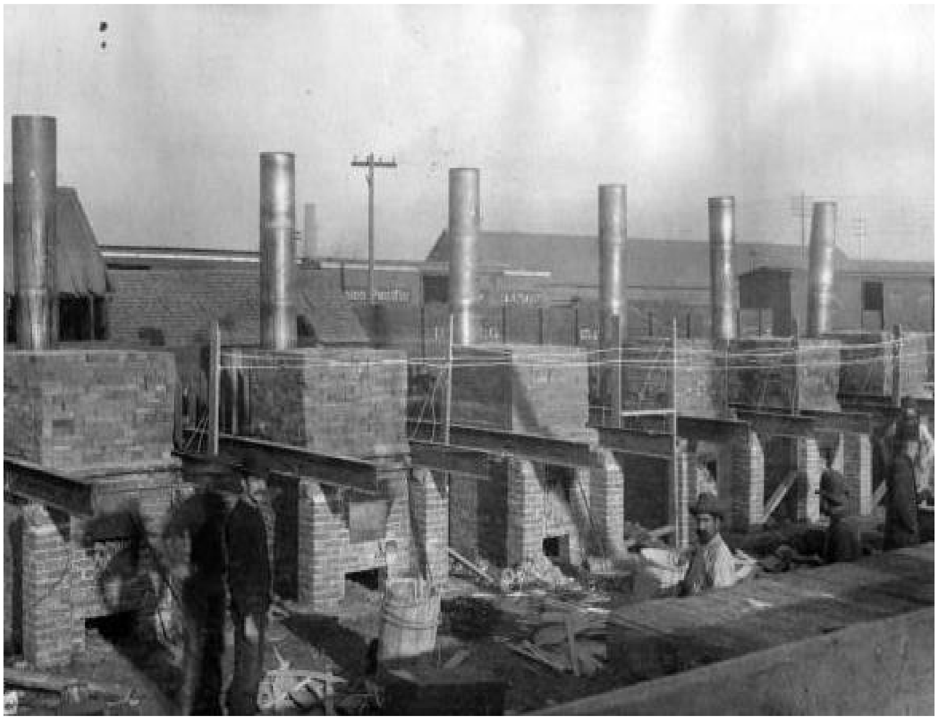

Figure 4a: Denver Equitable Building Fire Tests Pre-Test (as adapted from [25]).

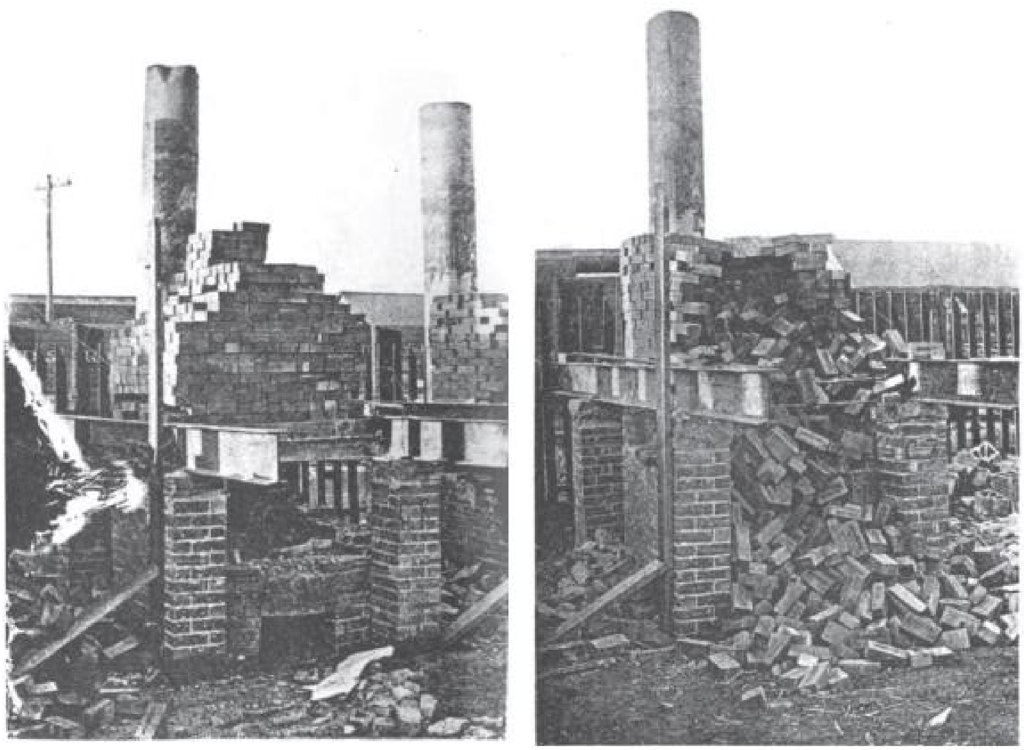

Figure 4b: Denver Equitable Building Fire Tests Post-Test (as adapted from [25]).

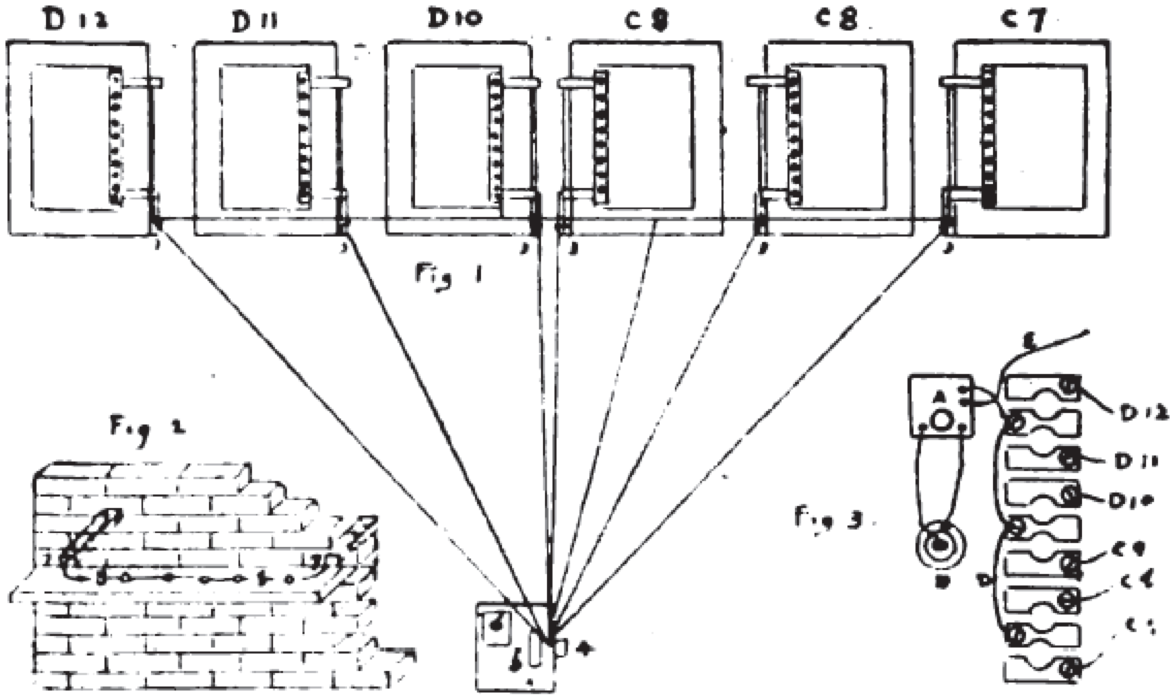

Plate 16. Apparatus for Temperature Measurements.

Figure 4c: Denver Equitable Building Fire Tests Instrumentations (as adapted from [25]).

4. A SIMPLIFIED NARRATIVE FOR THE STANDARD TEMPERATURE-TIME HEATING CURVE DEFINITION

In 1896, two very different test series were organized and reported by the New York Departmentof Buildings, which involved researchers from the Mechanical and Mining Engineering Department at Columbia University. It is well known that Ira Woolson is credited with the test series that eventually led to the standard temperature-time heating curve, or rather the Columbia curve as it is sometimes referred to [9, 10], but what isn't well known is the parallel research which was being done at Columbia in gas-furnace development at this time. The missinginformation in the existing narrative of the development of the standard temperature-time heating curve is the test series that was developed to utilize a controlled gas-furnace for variousbuilding elements. This was reported by mechanical engineer Sylvanus Reed in 1896 [24],predating Edwin Sachs' efforts in gas controlled element tests that used a range of fires for material assessment. The more well-known test series utilized a testing procedure similar to the aforementioned Denver tests, though now wood-stocked for specific floors [26].

Today, Sylvanus Reed's historical contributions largely mention his role in the creation of the modern metal aircraft propeller and note a relatively large absence in his career, with an apparent inactivity in the early 20th century prior to his aircraft propeller research. There is a lackof information recorded showing his contributions to define fire resistance testing and very little information about his contributions provided in existing references [9, 10]. It is important to consider Reed's contribution (correctly or wrongly) in his efforts to develop material element testing. Reed established principles were very similar to the ASTM standard temperature exposure that would be proposed by Sachs in 1914 and followed in 1918 led by Ira Woolson - aswell as some contemporary themes of fire severity argued today in element testing. Reed's fire resistance tests relied on using a gas fueled furnace to take advantage of the control of temperature. Reed documented various limitations for establishing test simplicity despite the broad objective of his test: "*steel or iron columns, girders, and beams, must be made on a full working scale and under the actual conditions, as far as possible, which would be obtained in a fire*" [24]. Reed went so far as to establish three different fire severities based on occupancy type(the fuel which would be expected in each that would control the severity of the fire), which wereestablished as the metric for this series. Reed argued that the tests' objective should be: "*To be a standard it must contemplate all fire possibilities, even the most remote, pertaining to those conditions….to establish a datum level from which allowable variations may be determined*" [24].This may be considered alike to the modern-day viewpoint of creating acceptable solutions. Thefire defined on the material would be controlled in a furnace as one of three cases: (1) 1371°C for six hours – warehouses; (2) 648°C for 1 hour – commercial store; or (3) 371°C for thirty minutes – office building or house. He does not detail how he arrives at these temperatures andhis text is filled with instances that demonstrate a provisional understanding of fire dynamics presenting a correct qualitative view of radiative heat, but a flawed quantified account of its calculation. These tests were performed under an applied service load, using a pyrometer to measure temperature, and his predominate concern was the testing of columns. Reed justified that all buildings should be expected to resist a conflagration, as to quantify what expected damage state would occur. This was to inform the insurance industry which is more interested inrecent discussions pertaining to fire resiliency. Reed's test program can be found documented inthe *Journal of the Franklin Institute* and is readily available to the interested reader today [24].

At this time, practitioners attempted to influence and debate the creation of these early tests and their merit when applied to real construction. Abraham Himmelwright (a practitioner who dealt primarily in developing concrete material systems during this time period) publicly advocated that "*The object of all tests of building materials should be to determine facts and develop results that may be of practical value in future designing. In order that such facts and results may have real value, three conditions are necessary: first, that the materials tested shall be identical with what is commercially available in the open market; second, that the conditions, methods, and details of constructions conform exactly to those obtainable in practice; third, thatthe tests be conducted in a scientific manner.*"[27] Himmelwright also stated that the design of structures had to resist thermal loading caused by fire: "*The actual and relative expansion of thematerials due to heat and deflections caused by unequal heating must receive careful consideration… The limit of safety is in some cases dependent upon temperature and in other cases upon expansion*." [27]

Of note, both Reed and Woolson studied under Frederick Hutton of Columbia University in theiryoung adulthood. Hutton was an expert in gas-furnace design [28] and it is natural to see that these researchers (Reed and Woolson) would eventually follow Hutton's combustion background influence in the development of their own test procedures [4]. While Woolson would start using wood stocked furnaces (it is documented he resisted gas furnaces due to their control difficulty and that they could not mimic the radiative heat seen in real fires), he would eventually advocatethe use of a gas furnace by the inception of the standard fire resistance test (see below). The tests performed by Reed were largely intended to be for informing the insurance industry and public interest. They were not meant for proprietary testing or material development, which is the clear distinction between how his tests evolved and how the tests of Woolson would later evolve to meet material competition. This, however, is not explicitly stated as the reason Reed'stests ceased (Reed's departure in developing fire resistance test philosophy correlates to the passing of his partner). The authors hypothesize that the lack of funding may have also contributed as a result of omitting the proprietary aspect of testing. It is interesting thatWoolson's tests were more aligned to ranking proprietary systems where the material manufacturer often paid to test their systems – Reed's materials were not financed, to the authors' knowledge, by the material manufacturers. Ira Woolson's tests would eventually form the basis of contemporary fire resistance test as defined by qualification (standardized) testing as per below. Ira Woolson oversaw the parallel test series [29]. Those tests considered primarilyflooring systems at first, with the original test criterion calling for a steady state temperature of 1093°C (the precision owing to conversions of Fahrenheit).

Although we credit Woolson in most historical papers (the 'Columbia' curve for example), careful examination will show that the test temperatures that Woolson originally defined were proposedin 1896 by the engineer Gus Henning, the chief engineer of the New York Department of Buildings [26]. Temperatures of over 1090 °C, were reached by feeding a wood fire furnace which was beneath the loaded flooring assembly and the duration of heating was meant to be held for over5 hours (Figure 5 illustrates schematics of the test hut used). Feeding rates were not specified but viewed as the speed necessary to reach the peak temperature as fast as possible. Post-test confirmation of peak temperatures would be performed that considered the presence of variousmetals that were confirmed melted.

In 1902, after the New York fire tests (1896-97), it was decided to specify a less severe fire exposure in terms of temperature. This new test standard [29] (a collaboration between Ira Woolson with Rudolph Miller) called for a sustained 'average' gas phase temperature equivalentto 927ºC (1700ºF) for 4 hours (with peaks still allowed to 1093 ºC (2000ºF)), hose stream cooling,and residual testing to higher loads (4 times the sustained fire service load) for a further 24 hours. If after this test, the floor's deflection (measured via surveying) did not exceed 1.4% of its span, the element was assumed to have 'passed'. The test still used a wood-stoked furnace since the thermal scenario was intended to be more severe than a real fire. In 1902, Woolson and Miller advocated in the New York Tribune, that "*no ordinary room would have enough inflammable material in it to maintain a 1700ºF fire.*" and that the basis for this heating regime was firefighters' qualitative experience in New York. In 1912, a complete catalogue of nearly 80 flooring systems tested (between 1896 and 1912) with this and previous test controls was

[4] The authors were unable to find any evidence that Hutton contributed directly or that his work was referred to inthe defining of the standard time-temperature heating curve.

published by Woolson in the proceedings of the International Association for Testing Materials conference in 1912 [29] (a precursor organization somewhat absorbed to the modern-day ASTM).

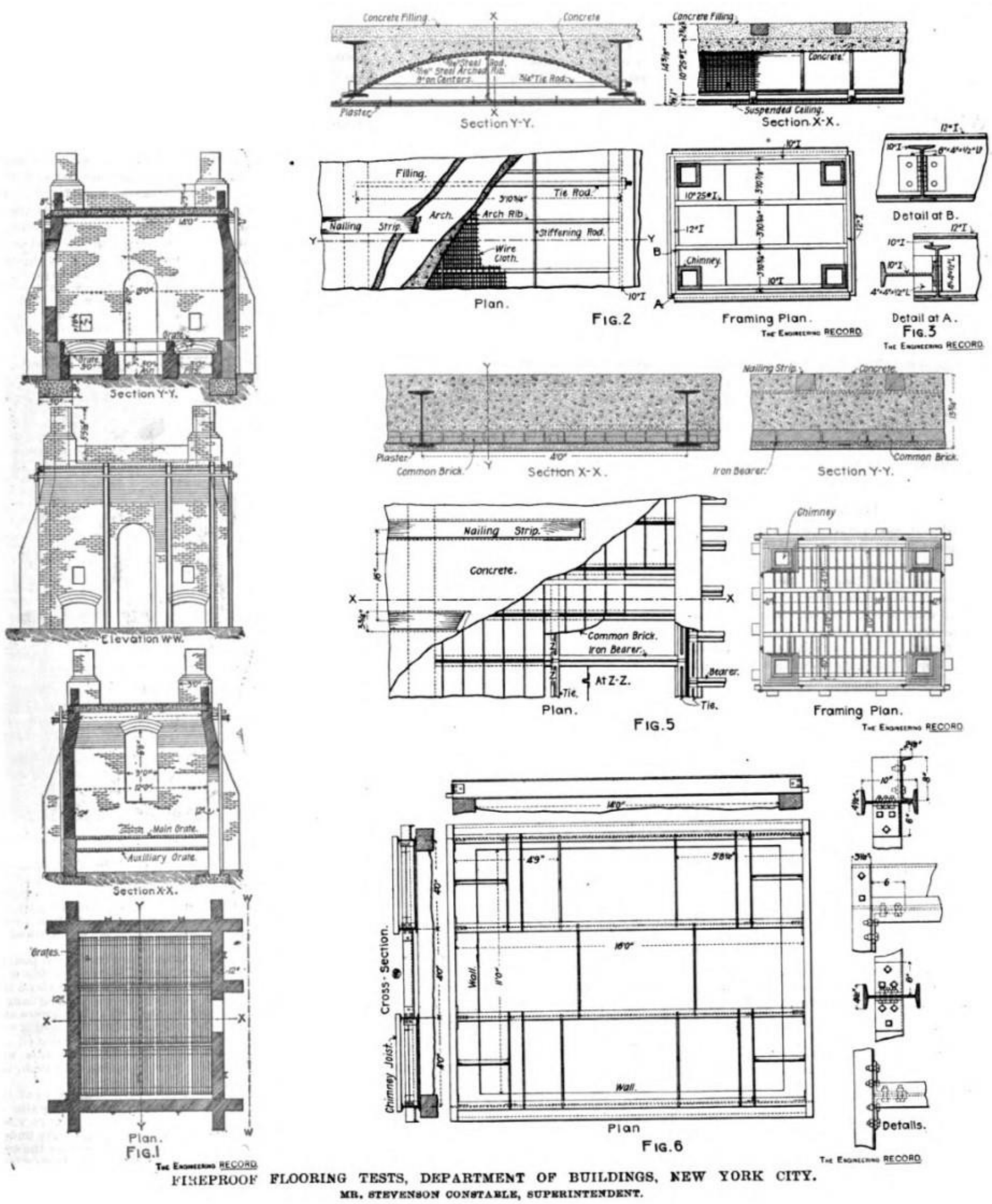

Figure 5: 1896-1897 New York Fire Tests by the Department of Buildings [26].

These standard fire tests by Woolson were often criticized during this period [30–33] because, atthis time, the floor tests were not nationally standardized and were not widely adopted outside of New York. They were also a subject of the Mazet Inquiry of 1899 which alleged corruption in the fire tests. Aspects of the alleged corruption and Mazet hearings are detailed by Wermiel (2007), where she notes that the Mazet hearings led the chief counsel and interrogator "*to conclude that all concrete floors were dangerous*", with the committee ultimately recommending"*a new Building Code Commission be appointed to revise the city's code*" [34]. For a small exampleof what was viewed as corruption, each test had to employ a night watch to prevent tampering of specimens for each test. These tests were followed by decreased influence of the city in the tests, and more control by research bodies to ensure their test integrity. In what appears timed to the response to the change in leadership in the momentum of the test series evolution, Gus Henning (see above) penned an open editorial in the New York Times where he publicly criticizedthe current test procedure being used by Woolson [31]: *"Other fakes I desire to call attention toare the fire tests now being made in New York City at temperatures of only 1700 °F. I herewith wish to declare fire tests of materials made at average temperature of 1700°F as shams and frauds. They do in no sense of the word determine the fire proof qualities of materials."* Henning'sreference to 1700°F (927°C – the one-hour mark used in the standard fire today), in the authors'subjective opinion appears in relation that real fires have more severity and that materials wouldbehave differently under this severe heating though the exact reasons for the public statement are not well documented in available literature.

Following criticism towards the New York building structure fire test series, various constructionmaterial agencies lobbied for change. This change was mobilized by Ira Woolson at the AmericanSociety of Testing of Materials (ASTM) as a new fire test standard evolved and was proposed in 1916 [32] (Figure 6). The test philosophy then had the intention to shift away from floors and toconsider columns (concrete, steel and timber). There exists no publicly available nor digitized documentation that explicitly defines the origins of the standard temperature-time heating curvethat was created in 1916 and still used today to assess fire resistance. Some have claimed (see [33, 35–37]) that in 1963, Bieberdorf et al. defined the curve's origins on the basis of metal melting points of metals [5], a theory that is common to hear in our practice, but has no evidence to the authors' awareness. The authors' personal examination of the Bieberdorf paper shows that it does not state this directly. Woolson himself does provide some commentary to the curve's origin being recorded as stating the curves intention as follows [38]: *"When you say it is a partition which will give two-hours' protection, it means it will resist a fire two hours accordingto the standard control curve given. That curve, which was presented last year (1917) purely as*

[5] Past historical papers on fire science have experienced these pitfalls. The historical review by Cooper and Stecklerprovides a factual critique of the standard fire tests origins [36]. They attempt to find the primary source documentwhich rationally explains where the curve comes from. They trace the origins from a secondary reference by a paperby Ryan which claims the fire's origins are in a paper written by Bieberdorf [33]. Cooper and Steckler were unable tolocate the Bieberdorf's paper to continue the search. That paper was found by the authors in the University of Edinburgh's BRE fire science library. The paper does not point to the origins of the time-temperature curve – it doesnot even reference an origin, it does confer upon the statement given by Shoub [37]. The study by Shoub indicates that it "... *apparently was based on temperatures found in the various stages of growth of actual fires in buildings using references such as the observed time of fusion of materials of known melting points.*" [37]. Both references donot provide a reference for melting points.

an arbitrary curve, has had a year's service by the Underwriters' Laboratories and by the Bureau of Standards at its laboratories in Pittsburgh".

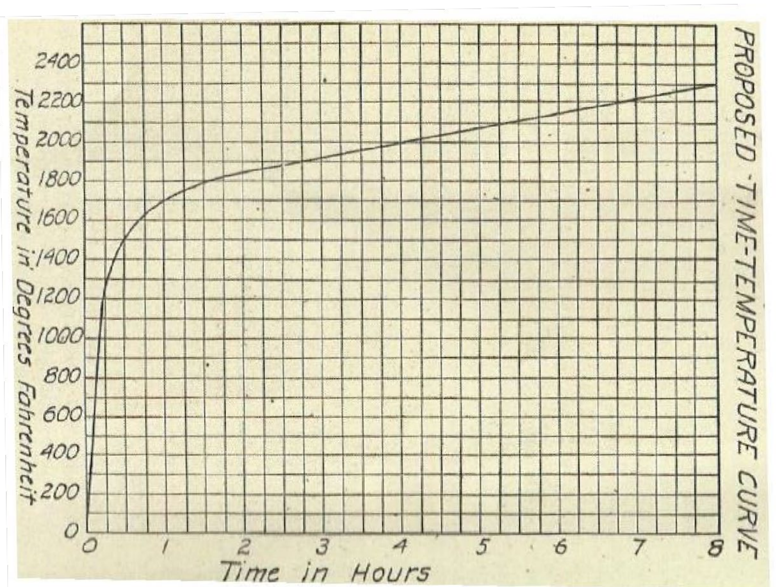

Figure 6. Time-Temperature Curve Proposed in 1916 [32].

The authors have prepared a subjective simplified interpretation of the time-temperature heating curve's origin (in the absence of other data available). The examination is of the test curveitself, previous linear set point standards, and the raw temperature data collected at the New York fire tests in 1896-97 (extrapolated from reference [26]). The temperatures recorded from these tests utilized a pyrometer which have experimental inaccuracies of +/- 300°C (evenacknowledged at that time). This is a subjective error as attributed to temperatures being recorded via human interpretation, as shown in Figure 7 as taken from reference [27]. The resulting comparison in Figure 8, illustrates what appears to be a linear linkage of the Henning 1896 and Woolson-Miller 1902 proposed time-temperature heating curve standards (a linear linebetween them at two set points up to 4 hours). Careful plotting of test data from the 1896-97 tests (see reference [26]) illustrates that the contemporary standard fire curve intercepts these points well, and achieves a linear fit between 1 and 4 hours of the Woolson-Miller curve adoptedin 1902 and the Henning Standard from 1897. This information is plotted in Figure 8 and requirescontinued research to definitively answer whether it is a best fit curve to test data and not real building fire data. Values are reported in Fahrenheit because they were measured in this unit atthat time. Barbrauskas and Williamson do a similar comparison, only showing all test curves usedprior to the 1916 standard proposal. They also indicate that the heating curve had seen no changein its defining plots since inception which carries true to today. The authors hypothesize herein that the standard temperature-time heating curve was developed along a more subjective rationale to link previously accepted time and temperature heating curves (specifically the Henning Standard and the Woolson-Miller Standard) and to ensure previous tests performed could still see acceptance under the new proposed heating curve. This hypothesis requires further scrutiny but nevertheless agrees with the currently available digitized literature from thattime period at the time of the writing of this paper.

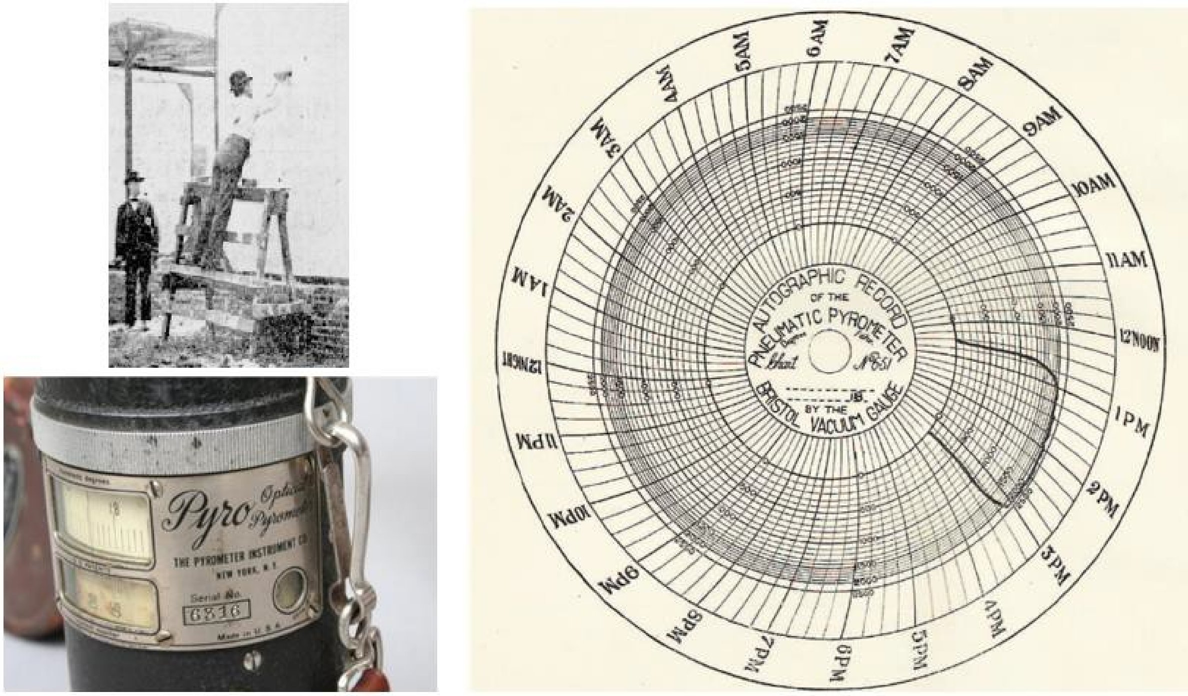

Figure 7. Pyrometer Temperature Recorded Measurement (see [27]).

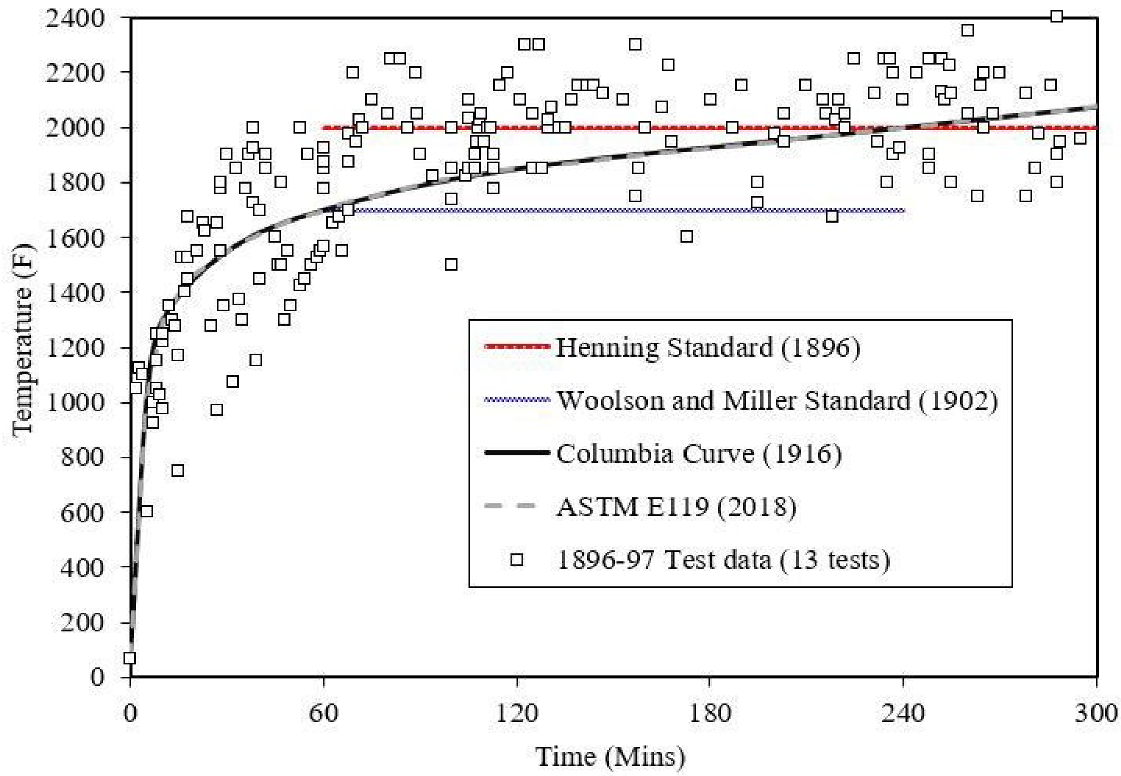

Figure 8. Author’s subjective Evolution of Fire Time-Temperature Curves as adopted by ASTM [39].

5. EUROPEAN EFFORTS TOWARDS STANDARDIZATION

Careful examination of literature will show similar initiatives for heating test control development, heavily influenced by architect Edwin Sachs, were underway in Europe at the sametime as Woolson. It is important to note the influence these efforts had on the standard temperature-time heating curve development. As aforementioned, the Universal Standard was originally proposed with modification as the ASTM standard time and temperature thermal boundary condition in 1914 (see Table 1). This standard heating control was considered for sometime in the Americas. That temperature condition was introduced at the International Fire Prevention Congress held in London in 1903 chaired by Edwin Sachs and his BFP committee. Alsoin attendance was an American delegation which included Ira Woolson. It was Woolson who would advocate the use of this temperature condition in ASTM meetings that would follow after1903. During these meetings, demonstrations of gas furnace design and use were also shown tothe American attendees. Woolson would report on the conference upon his return in brevity in 1904, however, the conference proceedings were more accurately published officially and in more detail in what was called the RedBooks [40]. The RedBooks periodical was internationally distributed (at times translated in French and German), and arguably the first scientific fire journal. These proceedings were summarized where relevant in Babrauskas’ work. Edwin Sachs health did not maintain in his later years and he passed away in 1919, after which his committeesaw little growth and disbanded. Figure 9 details the Redbooks and Edwin Sachs' gas-fired furnaces demonstrated at the 1903 conference. These photos are extracted from the Red Book conference proceeding from 1903.

Table 1. Edwin Sachs BFPC Universal Time and Temperature Exposure (advocated for ASTM usein 1914).

Classification	Sub-Class	Durationof Test at least (mins)	Minimum Temperature (°C)
Temporary Protective Class	A	45	816
	B	60	816
Partial Protective Class	A	90	982
	B	120	982
Full Protective Class	A	150	982
	B	240	982

After the conference, Sachs and Woolson continued correspondence and there are details of their meetings together aiming to develop a Universal Standard until at least 1912 [29]. The Universal Standard would be significantly criticized in North America and was not received well and subsequently not adopted. NFPA meeting minutes specifically targeted the standards more laxed criterion for its inclusion of a temporary building condition. While these meetings were notvery specific to why the standard ultimately was rejected, the lower severity of fire was critiquedin NFPA meeting minutes specifically due to implications to exit use where partitions would be present. ASTM would mobilize conferences afterwards (recorded as two meetings) to determinethe character of the standard temperature-time heating curve, and the familiar curve was then presented in NFPA Quarterly in 1916 [32]. Provisions of temporary, partial, and full were droppedand hourly ratings were recommended instead with one heating curve to be used [38].

After Sachs' death, momentum for developing standardized fire testing in Europe appeared slow,and it would not be until 1932 where the British Standards Institute revisited the subject. In thatyear, BS 476 was created which laid down the test procedure for assessing structural elements by means of a standard test, which adopted the time temperature heating curve from ASTM [41].It would not be until BS 476 was adopted in the 1930s, which largely mirrored the ASTM fire standard from that time in its initial conceptions but deviates today in test control [6]. BS 476 would later evolve into ISO 834.

Figure 9a. Edwin Sachs' RedBook Proceedings [40].

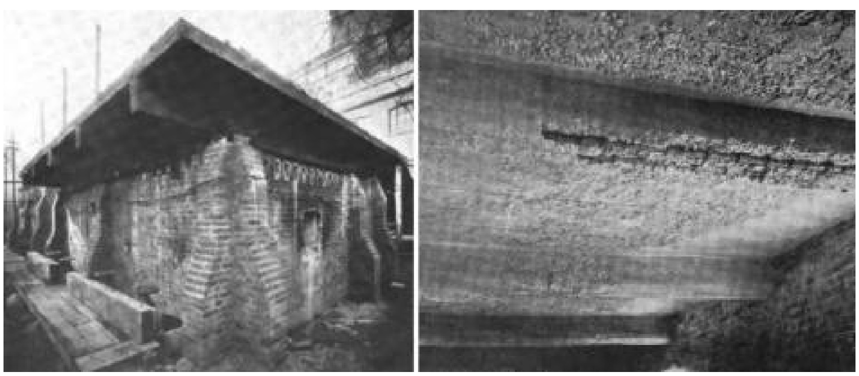

Figure 9b. Edwin Sachs' Gas Controlled Furnaces with Specimens [40].

[6] Today, the ISO 834 fire resistance test is specified to use plate thermometers as opposed to the ASTM E119 thatspecifies thermocouple. Both instruments govern the test control differently.

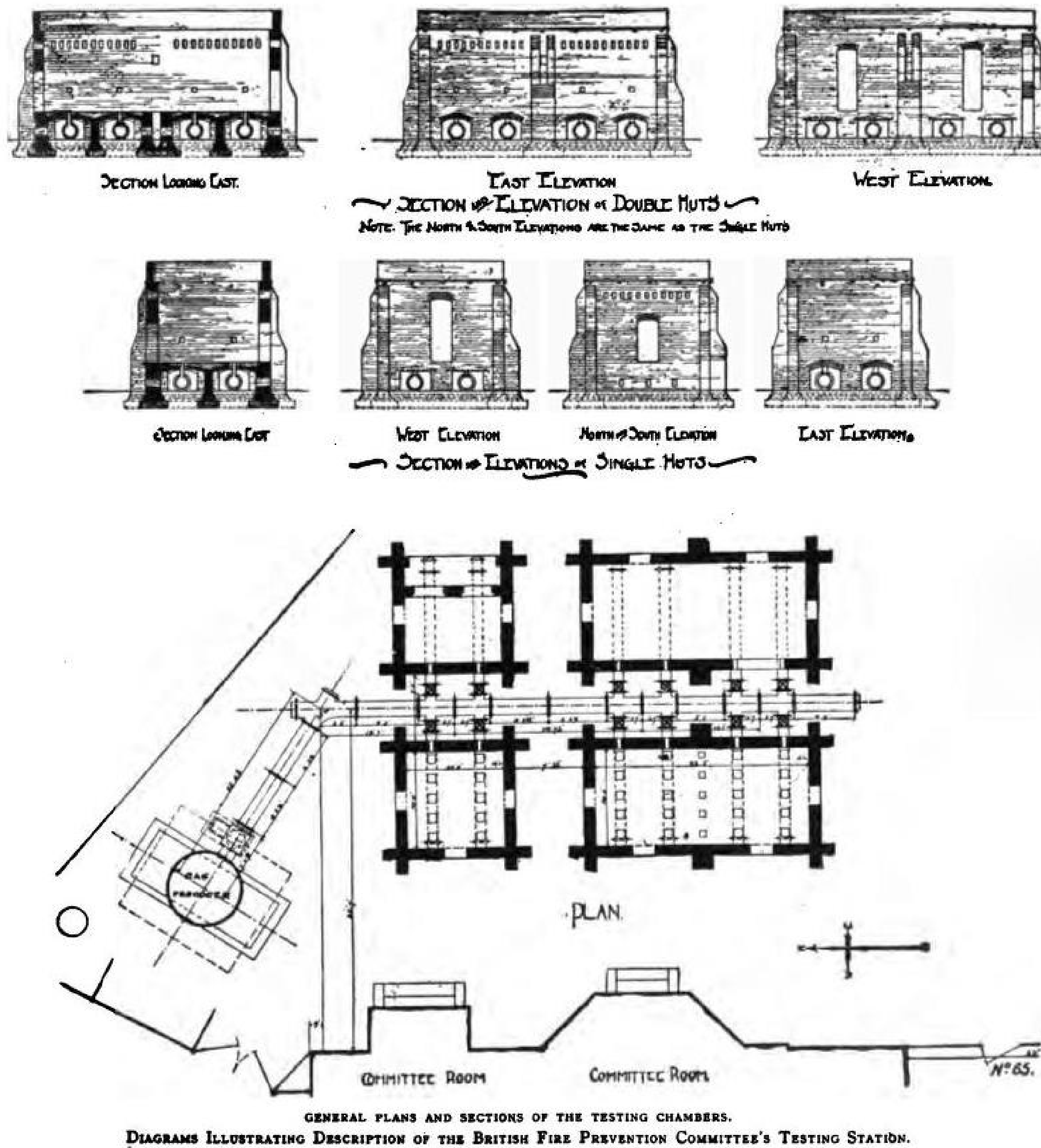

Figure 9c. Edwin Sachs' Gas Controlled Furnaces [40].

By the late 1970s and into the early 1980s, over-reliance on standard fire resistance testing was widely recognized as limiting innovation in architecture and construction, and technical papers began to appear which openly questioned the applicability of these tests. In 1981, practitioner Margaret Law remarked [1]: "*The standard temperature-time curve is different from the temperature time exposure likely to be encountered in real fires which will depend on the amountand type of fire load, the ventilation, size and shape of the building, and the activities of the fire brigade.*" Fire engineering researcher David Jeanes commented in 1982 [42]: "*although the traditional approach of assigning time for a given structural* *element or assembly allowed for a comparative measure between different types of construction; it is hard pressed to represent actual structural performance in a real fire due factors of restraint, redistribution of loads, moment resistance, as these are difficult to quantify and duplicate in tests."* It is the authors' opinion that the standard today recognizes fire resistance as the time duration that a 'mock-up' building element is able to withstand furnace heating based upon standard fire testing requirements and acceptance criterion defining test end.

6. AFTER THE TIME TEMPERATURE HEATING CURVE

The 1916 time-temperature heating curve was used for the first time in June 1917 to test a seriesof steel, and concrete columns; though six timber column tests were performed [43]. The criterion for the standard fire resistance test was then published by National Bureau of Standards (NBS) in the 1921 document: Fire Tests of Building Columns [43]. The test procedure used was very similar, albeit without technological control and procedural advances, to the modern ASTME119 fire test standard [8]. Overall, the tests considered using a controlled time-temperature heating curve on loaded columns using gas-controlled furnaces. Gas furnaces could better control the time-temperature heating curve in linear fashions. The tests were performed with manual control of temperature with consideration to temperature lag of the furnace and generally suggest poor resemblance to the standard time-temperature and heating curves. For example, Figure 10 illustrates the temperatures recorded for a concrete column test in 1917. For timber, temperatures exceeded 927°C after 30 minutes in most tests where timber was left exposed (temperature plots in the original reports are too poor quality to reproduce herein). In those tests series, six timber columns were tested (Pine and Douglas firs with measured moisture contents of 13-22%) with four columns tested without encapsulation technologies. The averaged charringrates of the unencapsulated members can be extrapolated (they did not report charring rates only time and char depth) to 1.13 mm/min which is on the higher end of what would be expectedfrom a modern furnace test controlled to contemporary ASTM standards.

The NBS documentation [43] does not describe the origins of the time and temperature heatingcurve, but it does provide comparison to versions of the Woolson-Miller Standard, US GeologicalSociety, and a version of the BFPC time and temperature heating curve illustrating that it envelopes each in that it is of higher peak temperature with time.

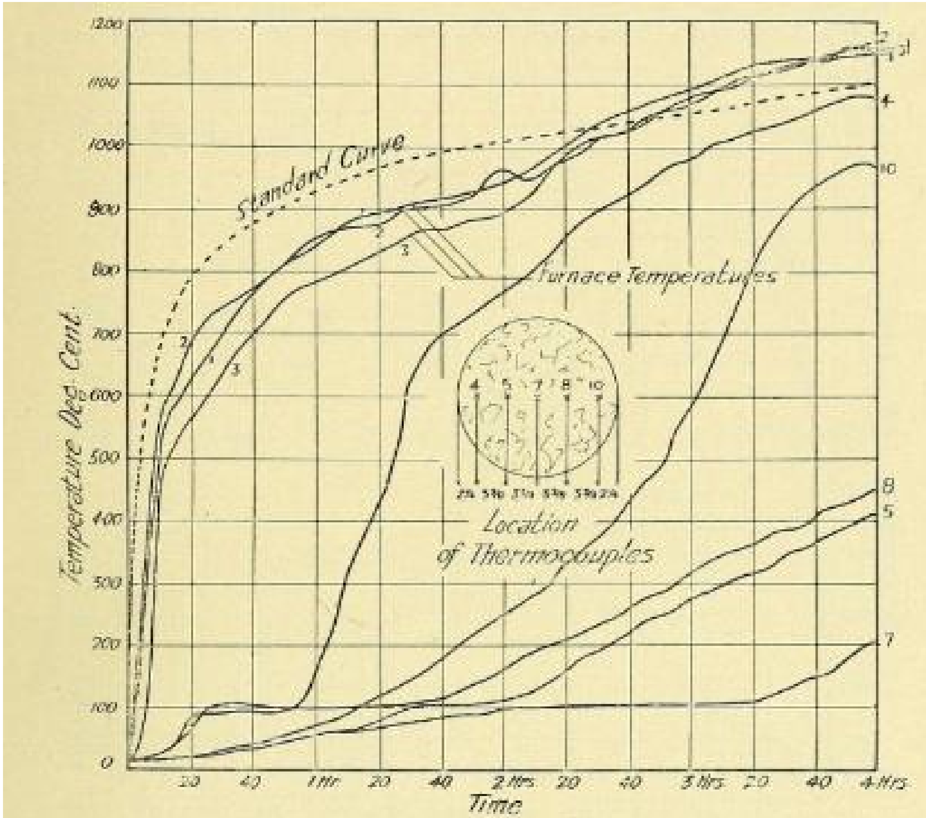

Figure 10. As-measured versus Control Temperatures of a Standard Fire Resistance Column Testcirca 1917 [44].

Even in the 1920s, it was widely known that the standard temperature-time heating curve was by no means indicative of a real fire. Simon Ingberg reported that "*it is necessary to assume maximum probable conditions both with regard to building contents and air supply, as consideredwith respect to intensity and duration of a possible fire. Compensations and adjustments between intensity and duration may be necessary under some conditions in order to approximate a fire duration having intensity equivalent to that of the exposure of the fire test*" [45]. Efforts principally by Ingberg [45] began to correlate a fire severity – using measurements from real burnout compartment tests – to the standard fire curve based on the "Equal Area Concept.". This concept was suggested in the aforementioned column test series above for which Ingberg contributed. Other researchers continued with the development of new concepts of equivalent fire severity based other severity metrics ("Maximum Temperature Concept," "Minimum Load Capacity Concept," and "Time-Equivalent Formulae"). Buildings could then be re-classified, not only by fire activation risk, but also by functions of fuel load, and 'equivalent' standard fire resistance times could then be specified for building elements. Minimal if any changes to the standard time-temperature heating curve were made through theyears in various iterations of ASTM standards that were produced after 1918. The test procedureitself showed increasingly less emphasis on residual capacity of the elements after a fire and to rather refine technological advances for test control (ensure more uniformity in heating for example). Overall, the fire community has largely followed the original testing procedure for construction materials and elements.

7. Contemporary Challenges and Conclusions

In recent times, concerns have been posed in the fire safety community regarding the degree ofheat provided in the fire resistance test. That being, the fuel provided to the furnace is reduced to compensate for the heat given off from the timber during the combustion process resulting ina reduced fire severity when compared to other building materials. Currently, in the interim of other approaches not being available, these challenges are addressed by fire safety engineers bylining the exposed timber areas of compartments, increasing the required duration of fire resistance tests (aiming to achieve an equivalence for the additional fuel load) and bespoke testing of timber frames. Additional research in timber fire dynamics is necessary to explore further any potential risks or opportunities.

There is no question that standard associations struggled and continue to struggle with these facts from creation of the time-temperature heating curve to contemporary times. In 1903, Ira Woolson when studying lumber specified the thermal boundary temperature simply to read 927°C as was per his criterion for the Woolson-Miller Standard [46]: "*This particular temperaturewas chosen because it is given by the New York Building Code as approximately the heat of a burning building*". There was no differentiation then for the thermal exposure varying between combustible and non-combustible materials by him at this time, because that was the state of the art for fire dynamics knowledge then – a subjective maximum temperature a building could see. For contextualization, Babrauskas [11] makes a statement that "*The current standard for firetesting reflects adequately the knowledge of compartment fires of 1918 but incorporates few of the later findings*". When the test was first developed, it was used for timber columns in 1917. Nowhere in the reports issued of those tests was the combustion effect on the test control discussed, yet examination of the report shows deviations from the standard temperature-time heating curve. Nonetheless, it was clearly recognized a decade later that timber had additional complications for fire testing, with the creation of various sub-committees within ASTM to undertake its study in fire. As the standard fire resistance test evolved, however, ASTM committee membership expanded, and funding was allocated for the study and standards creation for combustible materials such as lumber. This occurred in the late 1920s, followed by the creation of sub-committees to engage the issues of combustibility. In 1927, an ASTM meetingdiscussion emerged which began to question the credibility of the standard time-temperature curve for the assessment of timber (in fact all materials), "Standard Fire Tests for Combustible Building Materials" [47]. It was stated by Pierce of UL Underwriters: "*We do not know with exactness what are the temperatures characteristic of conflagration exposures nor do we know what are high and ordinary temperatures as applied to building fires. It is in such difficulties as this that the chief obstacles lie in developing the test methods on a scientific basis. We have our standard method of fire testing for materials; whether it is suited to the testing of combustible materials as well as non-combustible is certainly open to question. The method is somewhat arbitrary in that we apply a standard method of fire, standard rate of rise of temperature, to thetest specimen and observe the results without wasting time in discussing whether the temperatures and times involved are those that would exist in Louisville, Ky., or Chicago or Bostonin an actual fire. Our art-of-fire-prevention study has not reached that point*" [47].

Later in the same discussion, Simon Ingberg noted that "*I want to second what has been said here relative tothe necessity for proper interpretation of results. Using our regular fire testing procedure we areat present testing combustible or partly combustible constructions and obtaining certain results without any generally accepted interpretation as to what they mean when the constructions areapplied in buildings*" [47].

The discussion spoken of today [6, 7] regarding the time-temperature curve's usage on combustible construction is not new, but we do have better measurement tools to quantify it and investigate its implications [6, 7]. The temperatures given off by timber during a standard fireresistance test are not fully understood, nor are all building materials when real fires are considered as the field of fire dynamics is still evolving. It is merely that the standard temperature-time heating curve was framed as a unified gas temperature independent of any other fire because it was meant to be (at that time) the credible worst-case fire on concrete andsteel materials that were tested.

The nature of timber, and even concrete compartments, can create temperatures in real fires that exceed this time temperature heating curve, merely illustrating that the design philosophy of equivalence against this benchmark is a very debated topic that can polarize the approach to the design we are attempting to solve today. To suggest the standard temperature-time heatingcurve still serves its original purpose to this day is to argue that no advancements have been made in fire science, instrumentation, or even structural engineering since 1916 – which is not correct.

For example, calculated time-temperature design curves have been accepted and can be used indesign practice in recent years. These include Parametric heating curves [48], and more recently,there has been use and acceptance of the Travelling Fire Methodology [49, 50, 51] for the designof structures. These have been developed as an acknowledgement that the standard time- temperature heating curve is not representative of the current understanding of fires and cannot represent expected deformations of building elements as would occur in reality – particularly representing a cooling phase. These design curves have led to numerous contemporary and densely instrumented compartment fire tests both completed and planned. An arbitrary exampleof these heating curves is illustrated in Figure 11. These are shown with comparison to the standard time-temperature heating curve. These different curves used for design allow for more consideration of contemporary fire dynamics theory such as allowing for consideration of fuel and ventilation effects to name a few. These curves, as well as calculated realistic fires, in design are also continually being adapted by researchers and practitioners for structural fire engineering design. With consideration being on larger compartment spaces or even different construction materials beyond just concretes and steels.

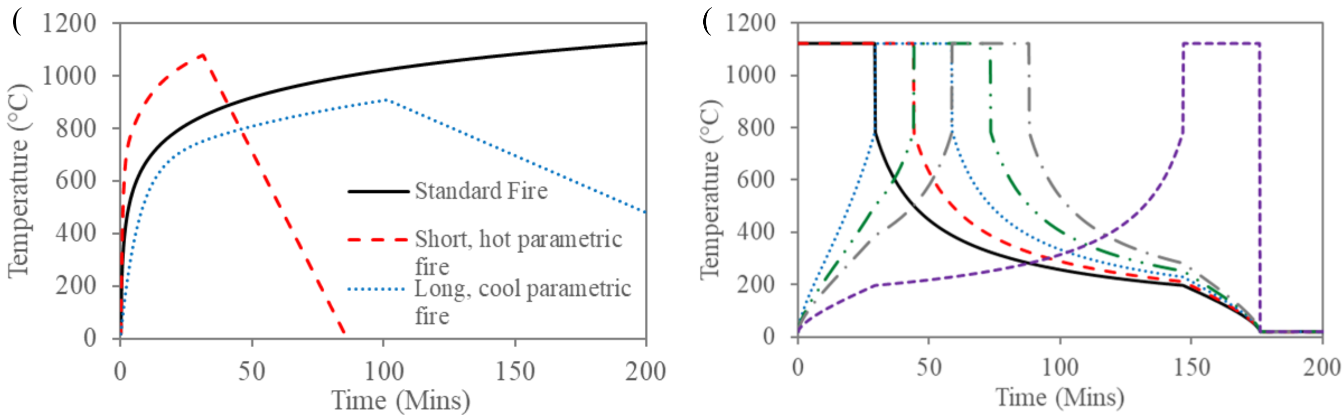

Figure 11. Different design fire examples including (a) the standard fire, two sample parametricfires and (b) a sample travelling fire as it progresses through a fixed dimensioned compartmentof 20 x 20m.[7]

We as an industry can respect the standard time-temperature heating curve's origins. But at thesame time, we need to use our contemporary knowledge of fire dynamics and structuralresponse to build upon it to create the next generation of standards. This review largely limits recommendations for the readership on steps forward other than that all practitioners should engage in the standardization meetings and discussion. The authors advocate that, if the standard curve is still considered unacceptable to some, it is of critical nature that thoseindividuals participate within the standardization process and convince these committees accordingly how upon they may improve the standard they have in question [52]. Such discussionand development will also have to show consideration to a range of building elements such as ducts, dampers, doors, fire stopping etc.

The authors have aimed to provide additional context to the historical narrative of the development of the standard time-temperature heating curve to add background and thethermal boundaries context for today's contemporary discussions. Herein, additional contributions from the original development cycle not distinctly covered by existing narratives were explored. The contributions to fire resistance testing from Sylvanus Reed are acknowledgedin a contemporary light and it is illustrated that the themes discussed were aligned to today's contemporary discussions. For example, the concept of examining building materials under a family of fires (different thermal boundary conditions to examine the most conservative design)for which, in some jurisdictions, we are returning to this concept in practice. This being in part due to acknowledgement that a structure may have more fuel load (contents or otherwise exposure to its combustible elements) and ventilation conditions to produce a thermal state that can evoke critical damage that a standard time and temperature heating curve cannot otherwiseevaluate on its own. However, unlike the past, the state of knowledge of structural and fire dynamics has improved where we know uniform thermal boundary exposures are not necessarily

[7] These calculated design heating curves utilize arbitrary inputs for illustrative purposes only.

the more critical heating to an assembly, where cooling (after heating) can be important for certain structural arrangements like steel connections or the invocation of cracking in concrete. The authors have also identified that no literature is (currently) available that supports a basis forthe time-temperature heating curve points. Practitioners will find discussion useful for current philosophical discussions pertaining to the time-temperature heating curve use for combustible construction.

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Appendix B:
Parametric Fire Curves

The analytical equation to calculate the fire temperature is divided into two curves, the heating and cooling curves, given in Eurocode 1 [70]. The heating curve is given as:

$$\theta_g = 20 + 1325(1 - 0.324e^{-0.2t^{\circ}} - 0.204e^{-1.7t^{\circ}} - 0.472e^{-19t^{\circ}}) \qquad \textit{(Eq.A1)}$$

$$t^{\circ} = t \cdot \Gamma \qquad \textit{(Eq.A2)}$$

where Θ_g is the gas temperature in the fire compartment (°C) and t is the time (hr). Γ is given as:

$$\Gamma = [O/b]^2/(0.04/1160)^2 \qquad \textit{(Eq.A3)}$$

$$O = A_v\sqrt{h_{eq}}/A_t \qquad \textit{(Eq.A4)}$$

where b is the thermal inertia of the enclosure boundary ($\sqrt{\rho c \lambda}$) (J/m^2 s$^{1/2}$ K), O is the opening factor of the fire compartment (m$^{1/2}$), h_{eq} is the weighted average of window heights on the wall (m), and A_t is the total area of enclosure (walls, ceiling and floor, including openings) (m^2). The maximum temperature Θ_{max} occurs when $t^{\circ} = t^{\circ}_{max}$, which is given as:

$$t^{\circ}_{max} = t_{max} \cdot \Gamma \qquad \textit{(Eq.A5)}$$

$$t_{max} = \max\,[(0.2 \cdot 10^{-3} \cdot \frac{q_{t.d}}{O}; t_{lim}] \qquad \textit{(Eq.A6)}$$

where $q_{t,d}$ is the design value of the fire load density related to the total surface area A_t of the enclosure (MJ/m^2) and t_{lim} is the limiting temperature based on the fire growth rate. After the maximum temperature has been reached, the cooling phase begins, given as:

$$\theta_g = \theta_{max} - 625(t^{\circ} - t^{\circ}_{max} \cdot x) \qquad \text{for } t^{\circ}_{max} \le 0.5 \qquad \textit{(Eq.A7)}$$

$$\theta_g = \theta_{max} - 250(3 \cdot t^{\circ}_{max})(t^{\circ} - t^{\circ}_{max} \cdot x) \qquad \text{for } 0.5 < t^{\circ}_{max} < 2 \qquad \textit{(Eq.A8)}$$

$$\theta_g = \theta_{max} - 250(t^{\circ} - t^{\circ}_{max} \cdot x) \qquad \text{for } t^{\circ}_{max} \ge 2 \qquad \textit{(Eq.A9)}$$

Appendix C:
Travelling Fire Methodology

In the improved Travelling Fires Methodology (iTFM), a travelling fire is a localized fire that moves within a large open compartment, composed of a leading edge and a trailing edge. The iTFM model assumes a non-uniform temperature distribution and that the fire travels in a one-dimensional direction through the compartment, at a constant spread rate along a uniform fuel load distribution. The size of the fire can vary depending on the fuel and ventilation parameters and therefore a family of fires (based on deterministic or probabilistic selection) is always considered to identify the most critical case when travelling fires are used in real design.

This model is assumed to be fuel controlled, as ventilation controlled fires are assumed to be unlikely to occur in large enclosures due to the large amount of available oxygen [71]. The main design variables were therefore determined to be the fuel load density, q_f, and heat release rate per unit area, $\dot{Q}$". These variables can be used to calculate the local burning time, t_b, which describes the amount of time required to burn out an area involved. This is given by Equation B1:

$$t_b = q_f/\dot{Q}'' \qquad \textit{(Eq.B1)}$$

The leading edge of a travelling fire is governed by the fire spread rate. The size of the fire is therefore limited by the range of spread rates found within experiments and real fires. The size of the fire, assuming the fire is the width of the compartment, can be found by calculating how far the fire would be able to travel before burning out at the ignition point, given by Equation B2:

$$L_{f,min/max} = s_{min/max} \cdot t_b \qquad \textit{(Eq.B2)}$$

where $L_{f,min/max}$ is the minimum or maximum fire length (m) and $s_{min/max}$ is the minimum or maximum spread rate (m/s). Rackauskaite et al. [71] gathered available data regarding typical compartment fire spread rates, with values ranging from 0.1-19.3 mm/s depending of the fuel source. These were identified as bounding values for valid fire sizes.

The iTFM model, through the concepts of leading and trailing edges, has a specified surface area of burning fuel, A_f, for a fixed time. This can be used to calculate the total heat release rate, using Equation B3:

$$\dot{Q} = A_f \cdot \dot{Q}'' \qquad \textit{(Eq.B3)}$$

This specified surface area will, however, change during the growth and decay phases of the fire. This changed surface area can be calculated using Equation B4:

$$A_f = L \cdot L_t^* \cdot W \cdot \dot{Q}'' \quad \textit{(Eq.B4)}$$

where L^*_t is the changing dimensionless fire size dependent on the leading-edge location $\dot{x}$ at the specified time t, L is the length of the compartment (m) and W is the width of the compartment (m). The following equations, B5-B8 respectfully, can be used to calculate the dimensionless fire size L^*, the fire spread rate s (m/s), the total fire duration t_{total} (s) and the location of the leading edge relative to the point of ignition $\dot{x}$ (m):

$$L^* = L_f/L \quad \textit{(Eq.B5)}$$

$$s = L_f/t_b \quad \textit{(Eq.B6)}$$

$$t_{total} = t_b(1/L^* + 1) \quad \textit{(Eq.B7)}$$

$$\dot{x} = s \cdot t \quad \textit{(Eq.B8)}$$

To perform the temperature analysis, the compartment is divided into two moving regions to calculate the non-uniform, transient temperatures: the near-field and the far-field. The near field is where the flames are impinging on the ceiling and the compartment experiences the peak temperature. The length of this region identifies the fire size, usually expressed in percentage of floor space or a flame spread rate. A conservative peak temperature was chosen to be 1200°C by Rackauskaite et al. [71] following an analysis of literature. A flapping angle, described as the angle from the main axis of the flame, is included to capture the possible fluctuation of the impinging flame. A value of ±6.5° was chosen for iTFM based on the experiments performed by Quintiere et al. [165]. The peak near-field is reduced due to flame flapping, because of the entrainment of cooler smoke into the fluctuating flame region. This equation is derived from Alpert's ceiling jet correlation by Rackauskaite et al. [71] with the equations shown below.

$$T_f = T_\infty + \frac{T_{nf}(2r_{x1}+L_f)-2T_\infty \cdot r_{x2}}{f} + \frac{32.28Q^{*\frac{2}{3}}}{H \cdot f}\left(r_2^{\frac{1}{3}} - r_{x2}^{\frac{1}{3}}\right) \quad \textit{(Eq.B9)}$$

Where f is the average temperature over the flame flapping length,

$$r_2 = f/2 \quad \textit{(Eq.B10)}$$

$$r_{x1} = max\left[0, r_0 - L_f/2\right] \quad \textit{(Eq.B11)}$$

$$r_{x2} = max\left[L_f/2, r_0\right] \quad \textit{(Eq.B12)}$$

$$T_{nf} = 1200°C \quad \textit{(Eq.B13)}$$

$$r_0 = \left(\frac{5.38}{H(T_{nf}-T_\infty)}\right)^{3/2} \quad \textit{(Eq.B14)}$$

The far-field is the smoke region which corresponds with the pre-heating or cooling of the structural elements. As the distance increases from the near field, the temperature of the smoke decreases due to the mixing with the air within the compartment. The far-field model is the analytical expression of Alpert's correlation with the limitation that it cannot exceed the near-field temperature. The far field gas temperature at a location *x* and time *t* can be calculated using the following equations:

$$T_{max}(x,t) = T_{\infty} + \frac{5.38}{H}\left(\frac{L\, L_t^*\, W\, \dot{Q}''}{x+0.5\, L\, L_t^* - \dot{x}_t}\right)^{2/3} \qquad \text{(Eq.B15)}$$

$$T_{max}(x,t) = T_{nf}, if \begin{cases} T_{ff} > T_{nf} \\ x + 0.5\, L\, L_t^* - \dot{x}_t \le 0.5 L_f \end{cases} \qquad \text{(Eq.B16)}$$

$$for \begin{array}{l} \dot{x} \le L \rightarrow \dot{x}_t = s\cdot t;\ L_t^* = \min\left[L^*, \frac{s\cdot t}{L}\right] \\ \dot{x} > L \rightarrow \dot{x}_t = L;\ L_t^* = 1 + (L_f - s\cdot t)/L \end{array} \qquad \text{(Eq.B17)}$$

Appendix D:
Guidance for the Post-Fire Structural Assessment of Prestressing Steel

From:

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ABSTRACT

Prestressing steel is commonplace in critical infrastructure like bridges. These structures are of significant societal and economic importance, and any disruptions affecting their service due to fire or other extreme events, can compromise life safety, structural integrity, and the economic cycle. The prestressing steel used in bridge infrastructure has been altered, chemically and mechanically, to attain high strength and durability properties, leading to complex behavior when exposed to high temperatures. The traditional guidance used by practitioners to assess prestressing steel post-fire performance are prescriptive charts that rely on the maximum temperature reached by the steel. These guidelines do not fully account for metallurgical changes introduced due to the time the steel was exposed to high temperatures, which has been shown in case studies to last up to eight or more hours in extreme conditions. Six prestressed bridge structures that were exposed to fire have been reviewed herein to discuss characteristic damage indicators and assessment methods. As part of the current work, multiple experimental programs are performed and other experiments by others are also reviewed, to analyse the residual condition of prestressed structures after fire. Non-destructive strength analyses coupled with calibrated hardness tests have been performed and compared to destructive methods with satisfactory correlation. Concrete exposed to high temperature was inspected, and research into the effect of exposure time on the post-fire strength of prestressing steel was examined. The results have shown that the current guidance relying only on critical temperature may not be conservative for long duration fires. Final remarks include suggestions for future research particularly for prestressed concrete and cable-supported structures, as well as recommendations to lower the critical temperature by at least 100℃ for current guidance to be applicable to longer durations fires.

1. INTRODUCTION

Prestressed structures are a popular contemporary construction, especially in critical infrastructure such as bridges. These bridges commonly use tensioned prestressing steel. Prestressing steel may take the form of strands or cables fabricated through using a series of wires that are high strength and cold drawn. Prestressed bridges may take the form of steel, concrete, or even wood structures. From the more common forms of bridge construction are prestressed concrete and for large spans of hundreds of meters, cable-supported structures. In concrete, prestressing steel tendons are tensioned before or after the concrete casting and released after the concrete has cured, creating an external compressive force on the concrete member. This process takes advantage of concrete's compressive strength and allows the structural member to sustain higher loads, achieve longer spans, and meet strict serviceability deflection limits. This also helps meet sustainability objectives by reducing the required concrete for construction. In cable-supported bridges (cable-stayed or suspension), each cable is tensioned to desirable levels to achieve similar goals.

Prestressing steel is highly sensitive to temperature changes which can invoke strength failure and other deformation characteristics. Some of the more prevalent concerns include the complex and rapid failure of the prestressing steel and unprotected stay-cables during fire and in particular in the case of prestressed concrete bridges, sudden strand thermal exposure if the concrete cover spalls [1]. Subsequently, post-fire investigations of critical infrastructure such as bridges must be conducted as soon as possible after a fire to assess structural integrity and limit severe economic losses. As previous studies have shown [2], bridge closures and failures due to fire and other events can be extremely costly as traffic must be redirected through over-congested or less capable routes. These investigations must be done accurately as any errors can affect public safety. The difficulty with post-fire investigations in prestressed structures is the complex behavior of the materials involved. Exposing prestressing steel to elevated temperatures can severely reduce mechanical properties and cause complex material changes related to its fabrication techniques. In addition, prestressing steel will not regain its mechanical properties as hot rolled steel does. As a result, a potential fire can have significant consequences in terms of post fire repair even if the event did not result in any collapse. Since the industry rapidly changes and refines these fabrication processes, prestressing steel is becoming more complex as a material, and understanding its behavior in high temperatures is becoming increasingly difficult. There is very limited guidance to assess the fire damage in prestressing structures. There is even less guidance for non-destructive assessment of structures containing it as this requires understanding the post-fire condition of the prestressing steel.

Different assessment methods have been adapted and applied to quantify the residual strength of prestressed structures post-fire and to determine the conservatism of the current guidance. Non-destructive strength analyses coupled with calibrated hardness tests [3] have been performed as well as research into the effect of exposure time on the post-fire strength of prestressing steel. Developing new guidance to assess and investigate prestressed concrete structures post-fire is required. This can be done by developing quick and effective non-destructive techniques for prestressing steel in critical infrastructure or by lowering the prescriptive critical temperature to be conservative. Due to the complexity of the behavior of prestressed concrete during and after fires, there is a need to determine whether a structure is safe after a fire, reduce closure time, and identify which repairs must be done.

Prestressed structures require further research to better understand their behavior when exposed to fire. This paper expands previous work [3,4] to illustrate the importance of understanding the post-fire material strength of prestressing steel, in particular for long duration fires. Herein, our focus is on prestressed concrete due to its popularity in construction and potential for long fire exposure, while also expanding insights for future research that can then consider cable-supported bridges. Subsequent studies for steel and timber are reserved for future research and beyond the scope of this current paper.

2. BACKGROUND AND MOTIVATION

Currents guidelines for practitioners by the Concrete Society [5] that assess prestressed concrete structures' post-fire performance use a graphical interpretation of the remaining strength that is dependant only on the peak steel temperature reached during the fire. This guidance has been widely used in the industry [6–8]. The severity of the fire, including its duration at peak temperatures, is not accounted for in that guidance. Relying on only the specific peak temperature for determining the remaining material strength provides a simple method for rapid estimation purposes but potentially fails to consider the effect of extended periods of high temperature exposure adequately. The peak temperatures can usually be determined by referring the heated concrete color to the guidance, which in turn is used to assess the remaining strength and load-carrying capacity. Despite its simplicity, a difficulty associated with this method is that its accuracy relies on qualitatively determining the post-fire concrete colour to determine steel temperatures [9]. Another possible method to identify the peak temperature is from a microstructural analysis of the concrete but this is time consuming. Literature reports 600°C as the most common critical temperature for prestressing steel, which the correlation suggests the remaining strength to be approximatively 50% of its original capacity after exposure to that temperature. Using a critical temperature approach is a common method in North America and Europe [6–8] and is the most convenient analysis regarding the post-fire damage assessment of prestressing steel. This method has its origins in the 1960s [10–12], which could be problematic since steel manufacturing has changed extensively since then. Modern fabrication techniques such as alloying have been shown to have an effect on the strength of steel with exposure to fire [13]. A number of tankers fires on highways have shown that these fires can last for several hours due to the quantity of combustibles involved and sometimes difficulty for the fire service to supress the fires. For potentially long thermal exposure conditions, when it is not certain that current guidance is adequate, the steel can be tested destructively. The problem in that situation is that the steel cannot be replaced or re-tensioned once removed from the structure. This leaves the structure without the beneficial effects from the pre- or post-tensioning. Therefore, a more robust method to conservatively assess the residual strength within the prestressing steel post-fire is required to ensure the safety of critical prestressed concrete infrastructure. This is even more relevant for bridges that their post fire repair and maintenance is more complex and complete reconstruction is often difficult. This work presents guidance on undertaking such assessments. This is achieved with particular supporting literature from previous research programs by others and post fire assessments of bridge structures exposed to fire as discussed below.

2.1 POST FIRE ASSESSMENT RESEARCH PROGRAMS

When it comes to assessing an existing prestressed structure, there are certain methods that can be used to determine the residual prestress force. Bagge, Nilimaa and Elfgren [14] performed an extensive experimental program that aimed to calibrate and further develop the existing non-destructive assessment methods. Some of these methods discussed were also used in the assessment of prestressed double-tee beams that were exposed to fire by Masetti et al. [15]. Within this study, the use of non-destructive and destructive testing was outlined for use in evaluating prestressed concrete after a fire, in combination with an analysis of the fire intensity, duration, and created temperature profiles. These methods were used to assess the residual strength and prestressing losses within the concrete structure affected by the fire.

Wu et al. [16] conducted an experimental program involving two types of bonded post-tensioned concrete bridge beams (box and tee) exposed to hydrocarbon pool fires. The analysis identified the losses of load carrying capacity of the beams, and the tests illustrated the large deflections caused by thermal creep after the loss of stiffness of concrete which results in the rebound within the prestressed members. Another study of a full-scale single span prestressed concrete bridge exposed to a pool fire by Beneberu and Yazdani [17] further identified the high risk of typical bridge girders when exposed to hydrocarbon pool fires.

Bamonte and Felicetti [18] have previously examined the behavior of prestressed concrete members (I-section and double-tees) under various load levels when exposed to up to 120 minutes of the ISO 834 fire with a cooling branch with rates of 3-10°C/min. The analysis identified the need to examine the cooling portion of a fire as the peak temperature within the prestressing steel is often reached after the onset of cooling, which can lead to delayed failure. The researchers also showed that fast cooling rates are more beneficial when it comes to preventing steel reaching peak temperatures, however this needs to be balanced with limiting the thermal gradients within the concrete to prevent spalling.

Previous work by Zhang et al. [19] has examined the deterioration of prestressing steel at elevated temperatures and after cooling to create empirical formulae. Their analysis consisted of testing central wires from seven-wire, grade 1860 prestressing strands, conforming to GB/T 5224 and to BS 5896, to represent steel from mainland China and Europe respectively. Two testing methodologies were employed to examine the tensile behavior at high temperature (in fire) and after cooling, and the results were then compared to unbonded post-tensioned two-way concrete slab tests. Zhang et al. [19] proposed empirical formulas for the degradation of Young's modulus, yield strength and ultimate strength for both tensile conditions examined.

2.2 POST- FIRE PERFORMANCE CASE STUDIES

There have been multiple recorded major fires involving prestressed bridges within the last 20 years. Two structural types are considered, those of prestressed concrete and cable-supported, the latter being a less frequent bridge type with fewer case studies to draw upon but critical to emphasize that the fire exposure risk is credible. In reference to the prestressed concrete bridges, the prestressing steel in all case studies referred herein was bonded concrete, meaning the steel tendons were directly bonded with the concrete and grouted in the sheathing. This is opposed to unbonded prestressing in which the steel tendons are greased and have no mechanical compatibility with the concrete. Prestressed concrete bridges often use bonded prestressing because it simplifies the pre-fabrication of concrete girders. Careful attention is given to the varying fire characteristics, fire assessment, and the key conclusions in each of the case studies.

2.2.1 PRESTRESSED CONCRETE

Puyallup River, USA, 2002

In 2002, a railroad tanker collision occurred in Puyallup River, USA [20]. The collision released and ignited the on-board hydrocarbons, resulting in a fire with rapid temperature increase and high peak temperatures that lasted for over an hour. The post-fire assessment followed similar protocols outlined in by the TR 68 [5]. The concrete color, deflection behavior of the bridge spans, spalling, and the residual prestressing steel strength were considered. The engineers used a variety of destructive technologies to meet the objectives including sampling concrete cores and extracting prestressing steel strand samples. The remaining prestressing was assessed using a dial reading caliber to calculate the remaining force. The key conclusions of the engineering report [20] mentioned the amount of stress relaxation observed in the prestressing steel as mostly small even though there was severe damage in the form of spalling and there were indications of the steel being heated to nearly 500°C. It was also suggested that there is a limited amount of experimental data and literature that analyze the effects of hydrocarbon fires under bridges. Even though the steel was determined to retain its 'original' material properties, the girders were still replaced since repair costs were the same as replacement.

Don Valley Parkway, Canada, 2008

The second case study examined occurred on the Don Valley Parkway in Toronto, Canada [6]. This fire occurred on September 4th, 2008 when a vehicle crashed into a pier, causing a fire that lasted over three hours. The post-fire assessment followed the protocol outlined in Ontario's Ministry of Transportation (MTO) guidelines. The engineers noted the presence of longitudinal cracking, spalling that exposed prestressing steel, and a pinkish concrete colour. The available report regarding the accident is vague with regard to the immediate post-assessment of the remaining stress and strength capacity of the prestressing steel. However, the report does detail that repairs were undertaken shortly after the accident by pouring a new 200 mm thick concrete slab on the soffit to reinforce badly damaged areas of the existing deck as part of an emergency response plan. A new transverse diaphragm was also cast between both ends of the new slab. The bridge was heavily instrumented after the fire by means of strain gauges and deflection transducers to monitor repair quality. A later assessment was carried out indicating that the structure had an elastic response and full recovery under cyclic truck loading. The report also indicated that as there was no significant deflection after fire, the prestressing tendons were not affected by the heat, and that the concrete temperature was in excess of 600°C on the basis of colour alone. The report cautioned that the bridge could experience accelerated deterioration in the future and insisted upon studying the long-term effects on the bridge post-fire whilst highlighting the need to rapidly repair cracking and delaminated concrete to prevent corrosion or durability issues. The previous case study outlined similar concerns with regards to potential corrosion implications to prestressing steel if concrete was not repaired appropriately. A second repair was done in 2009 where some concrete was patched and six girders in the main span were wrapped with carbon fibre reinforced polymer (CFRP). Six years after the first load test, the bridge's performance was reanalyzed on November 23rd, 2014. The second load test was comparable to the first. This could likely be attributed to the intermediate repairs between tests.

Deans Brook Viaduct, United Kingdom, 2011

The Deans Brook Viaduct fire that occurred in 2011 in the United Kingdom is said to have started in an alley and scrapyard near the bridge [21]. The fire brigade reported that the fire caused the temperature under the bridge to reach up to 800°C. Spalling was observed three hours after the beginning of the fire. The bridge condition was assessed using a number of guidelines, including TR 68 and a variety of Highway Agency reports that imply that prestressed concrete beams retain the majority of their initial prestress after being exposed to a five-hour fire (the severity of that fire was not specified and prestressing levels were inferred from dated standard fire tests – see reference [1]). Since the bottom layer of prestressing steel within the beam was exposed due to spalling, the initial assessment hypothesized no strength remained in the bottom layer and 75% remaining strength in the upper steel layer. The post-fire assessment included hammer tests to assess delamination, destructive testing considering aggregate discolouration, and microscopic analysis for concrete micro-cracking. A section of the prestressing steel was removed to perform hardness and tensile strength tests. It was concluded that the TR 68 guidance was acceptable while noting that no significant tension losses (less than 15% loss) in the prestressing steel were present where the concrete cover remained. Where the cover was completely lost, the exposed steel had significant losses in tensile strength greater than 60% loss). It was highlighted within the engineering reports that there is a need for a standardized test procedure to determine the remaining tension in the steel after being exposed to fire. The current methods use destructive testing, by cutting the steel and measuring its relaxation, or by vibration analysis.

Atlanta Georgia, USA, 2017

One of the most recent major fires involving prestressed concrete bridges occurred on March 30th, 2017 in Atlanta, Georgia, USA [22,23]. The fire started underneath the Interstate 85 bridge within a dumpster that spread to nearby high-density polyethylene (HDPE) pipes stored under the bridge by the State of Georgia after a project was halted in 2011. The fire lasted approximatively two hours, but a bridge section with five prestressed concrete beams collapsed after the first hour. The concrete cover completely spalled before the collapse, exposing the prestressed steel to the fire. The materials stored under the bridge were claimed to be non-combustible and required long exposure to high temperature for ignition to occur. After the collapse, 350 feet of highway in both directions needed replacement. In addition to the direct financial cost of \$16.6 million, the reconstruction of the bridge required commuters to find alternate routes and caused extra strain to be applied on the city's transportation infrastructure. Since the Interstate 85 bridge was vital to reducing traffic congestion, construction on other road sites was stopped to focus on rebuilding the bridge. Local businesses near the construction site reported lower earnings as the traffic within the area was reduced. In response to this fire, the State of Georgia, as well as many other states, started reviewing their storage policies. Around thirty states inspected their storage underneath bridges.

2.2.2 CABLE STAY BRIDGE CASE STUDIES

Mezcala Bridge, Mexico, 2007

A vehicle fire involving two school busses and a heavy goods vehicle (HGV) carrying coconuts occurred on the Mezcala cable-stayed bridge in 2007 [24]. The fire was adjacent to a cable-stay encased in an HDPE sheath which ultimately failed as a result of the heating; limited damage was also inflicted on an adjacent cable [24]. Traffic on the bridge was closed until the cable was replaced. This fire event raised concerns pertaining to the potential progressive collapse of cable elements during bridge fire scenarios, especially those with HDPE sheaths which can exacerbate fire scenarios by further contributing to the fuel load and flame spread. This was observed two years prior on the Rion Antirion Bridge in Greece where lightning initiated a fire of a HDPE cable sheath that lead to the failure of a cable, which further resulted in damage to an adjacent cable. This bridge did not reach full operational capacity for two months during the replacement of the two cables [25].

New Little Belt Bridge, Denmark, 2013

In 2013, a collision of an HGV on the New Little Belt Suspension Bridge in Denmark created a fire lasting approximately 30-45 minutes at the lowest point of the main cable [25,26]. Although minor damage was observed on the 580 mm diameter main cable, a post-fire analysis observed only an 8% decrease in ultimate strength and no decrease in yield strength [25]. Firefighters estimated the fire exposure temperature at the main cable to be approximately 500°C. Despite the relatively minor damage to the main cable, a suspender cable adjacent to the fire required replacement and the galvanized coating of a nearby cable band was observed to have melted [25]. Following this fire, an operational risk assessment was conducted to assess potential fire risks to the bridge and found that, despite the low traffic of gasoline and fuel tankers, fires from heavy goods vehicles (HGVs) carrying flammable (but not necessarily hazardous) goods were sufficiently significant to warrant additional fire protection measures as they can still achieve hydrocarbon-like temperatures of over 1100°C [26]. Ultimately, it took four months for the bridge to fully reopen to traffic [25], emphasizing the sensitivity of cable-supported structures to fire scenarios. The following year in 2014, as will be discussed with an analysis of stay-cable strength reduction in Section 5, the Chishi (Red-Stone) Bridge in China experienced significant damage and the loss of nine stay-cables as a result of a fire that spread primarily through the HDPE cable sheath system [27].

2.3 CONCLUSIONS FROM THE AVAILABLE LITERATURE

Time is an important factor when dealing with critical infrastructure post-fire. Prestressed structures need to be quickly assessed and the severity of the fire damage determined. As was the case in the Interstate 85 collapse, closure for extended durations can cost millions in economic damage. Within these case studies, it is shown that there is a lack of standardized procedures for assessing the remaining prestress within a concrete bridge or any prestressed structures. The subject of assessing concrete structures post-fire from an international standpoint has been re-addressed by Rush et al. [28], however, it still requires that the applied fire condition be fully quantified. Prestressed concrete bridges can be exposed to varying lengths and severities of fires, as demonstrated by the multiple case studies. This leads to a diverse range of potential fires and damage indicators within this type of structure, causing framework development for analysis to be quite complex. It would be beneficial for the framework to analyze all manners of prestressed concrete structures post-fire (buildings as well as bridges) to inform its development. From the case studies, it appears that spalling beneath the prestressing steel is always prevalent even with lower severity fires. This emphasizes that localized damage to the prestressing steel can have a predominant effect on the structure's strength. The important aspect is to properly assess the remaining capacity of the prestressing steel with little invasion as the condition of the structure is highly dependent upon it.

From these case studies discussed in the previous sections, it can also be observed that there is a trend towards destructive tests or time-consuming microstructure analyses. While current technologies may be capable of assessing traditional structures post-fire, prestressed structures have a higher complexity and these methods are too approximate or time-intensive. Concrete can be analyzed considering the colour change that occurred, however there is doubt that this can be used to determine the peak temperature in the prestressing steel as the steel can conduct the heat longitudinally [2], resulting in a lower steel temperature than the concrete surrounding it. There have also been experiments that showed that 'non-pink' concrete reached temperatures that could be considered critical for prestressing steel [29].

3. EXPERIMENTAL PROGRAM

While there are many factors that could be considered, the experiments discussed herein consider prestressing steel strength after being exposed to fire for a range of time durations to produce conservative and conventional guidance. All tests discussed within this section were conducted over the course of several years utilizing the same prestressing steel stocks as procured from a mill in the United Kingdom and a Mill in Singapore, fabricated to the BS 5896 and AZ/NZS 4672 standards respectively. The chemical composition of these strands, which is important to describe high temperature resistance among other characteristics (corrosion for example), are described in Table 1. Research herein follows the authors previous study of prestressing steels exposed in fire for which the same steel stocks were used where stress relaxation, and strength loss are considered (see [13]). The study herein considers the post-fire structure acknowledging that steel will experience a partial recovery in mechanical properties post–fire after the structure cools back to ambient.

Table 1. Chemical composition as a percentage mass of the prestressing steels considered herein.

Element	BS 5896	AS/NZS 4672	Historic
C	0.88	0.79	0.79
Cr	0.01	0.29	Not measured
Mn	0.61	0.59	0.78
P	0.0070	0.012	0.012
Si	0.26	0.28	0.19
S	0.019	0.008	0.031
Ni	0.02	0.03	Not measured
Cu	0.01	0.14	Not measured

There are three test series considered herein, two generating specific post-fire strength data, and a third test series to verify the findings herein. Overall, the experiments were conducted to determine a more time efficient method of assessing the residual strength of the prestressing steel, while revising guidance to ensure it is conservative.

3.1 PRESTRESSING STEEL STRENGTH TESTS

The effect of fire exposure duration on the remaining strength of prestressing steel post-fire was examined. Two different prestressing steel tendons types were used as aforementioned. Once received, the core wires of the strands were extracted as these could be considered without the influence of residual stresses (which should be investigated in their own in future studies), and in order for conventional equipment could grip the material wires where outer wires will have difficulty in gripping. The diameters of the wires were 4.15 and 4.4 mm respectively.

The prestressing steels considered had similar mechanical behaviors at ambient temperature, with an ultimate strength of 1950 MPa and rupture engineering strains of 7% elongation. Testing during the fire has been considered elsewhere [13] and has shown that this stock of AS/NZS 4672 steel had a higher residual strength capacity at high temperatures. The steel samples were heated and allowed to cool to ambient, then were subjected to tensile testing. Microscopic analysis and hardness testing followed (see Section 3.2).

The steel samples for the tensile testing were cut in lengths of 25 cm which provides a suitable length for strain measurement in accordance to standard tensile testing methods. Target temperatures were reached using an annealing furnace, with temperature intervals of 100°C up to 900°C, apart from 727°C which is explicitly considered as the eutectoid strength level. The samples were heated at a rate of 10°C /min until they reached their specified peak temperature, and then held for dwell times of two, four, or eight hours. The heating was applied without interruptions and without any load applied to the samples. To ensure accurate thermal exposure, K-type thermocouples measured the temperature at the surface of the samples. These thermocouples were used by the researchers as the accuracy of the K type thermocouple was lower in comparison with a thermocouple tree within the heating unit, with an expected error of a K type thermocouple (+/- 2°C). A limited amount of AS/NZS 4672 steel was available (only a specific amount could be shipped to the authors from Singapore), therefore only selected combinations of temperatures and times were tested. For a complete test heating program, see Table 2. All the samples were then cooled to ambient in air. Different quenching techniques were investigated, including ice baths and cooling in the furnace, but no significant difference in the residual strength was observed from these methods.

Table 2. Test heating program.

Temperature(°C)	Duration of heating			Prestressing steel
	2hr	4hr	8hr	
20	3	0	0	BS 5896 AS/NZS 4672
100	3	3	3	BS 5896
200	3	3 0	3	BS 5896 AS/NZS 4672
300	3	3	3	BS 5896
400	3	9 0	3	BS 5896 AS/NZS 4672
500	3	3	3	BS 5896
600	6 3	3 0	3	BS 5896 AS/NZS 4672
700	3	3	3	BS 5896
727	3	0	3	BS 5896
800	3	3 0	3	BS 5896 AS/NZS 4672
900	3	3	3	BS 5896

Note: Additional tests were performed for microscopy and hardness analysis that are not included in this table.

For the tensile testing, a uniaxial tensile loading actuator was used with a loading rate of 2 mm/min. The tests were instrumented using digital image correlation (DIC) which was used in combination with the GeoPIV RG software [30] to determine the strain, true stress, and ductility of the prestressing steel. Figure 1 shows the loading apparatus with the camera used for the image correlation (not shown is the halogen lighting used as part of the DIC procedure). To obtain accurate measurements, the specimens were speckled with black and white paint which allows better point tracking on the DIC software. A stationary camera, in this case a Canon 5D Mark 3, takes a picture at a specified frequency, chosen to be one every five seconds. The software can then track pixel movement between sequential images to determine strain and deformations such as cross-sectional reduction. The Digital Image Correlation technique has been tested for accuracy in previous studies, and the method used for this experiment can be found as described in references [13, 31, 32]. The prestressing steel samples were tested following the best practice methods for DIC [32].

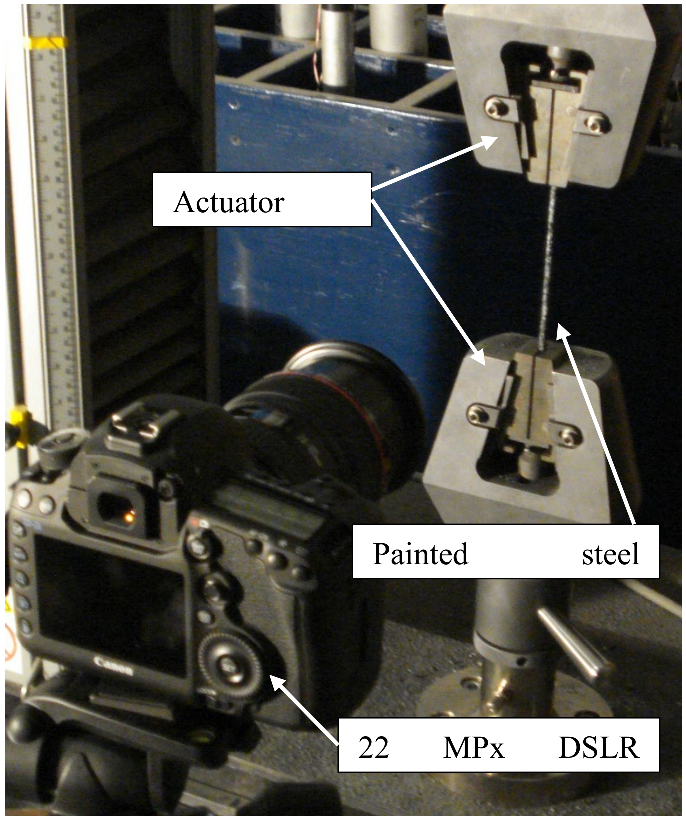

Figure 1. Loading apparatus with digital image correlation.

Figure 2 illustrates the stress-strain relationship for a BS 5896 steel sample exposed to different temperatures for two hours. Included in this figure is the true stress, calculated using DIC to determine the reduction in cross-sectional area. There is very limited data regarding the failure strains of prestressing steel post-fire or of the true stress of this material under loading. True stress is important if the prestressing steel is not replaced within the structure post-fire as it helps determine the overall failure characteristics of steel.

Figure 3 shows the failure profile of the steel at varying temperatures. It can be seen that the cup and cone profile stops once the temperature exceeds 700°C when the steel becomes more brittle and the area reduction is minimal. True stress is also important for understanding the ductility of the prestressing steel, which is essential if the steel continues to be used post-fire. If the steel reaches temperatures exceeding 700°C, the brittle behavior means there will be less warning before failure. For this reason, it is important to limit the strength losses to 50%. The strength gained from high temperature exposure cannot be relied upon.

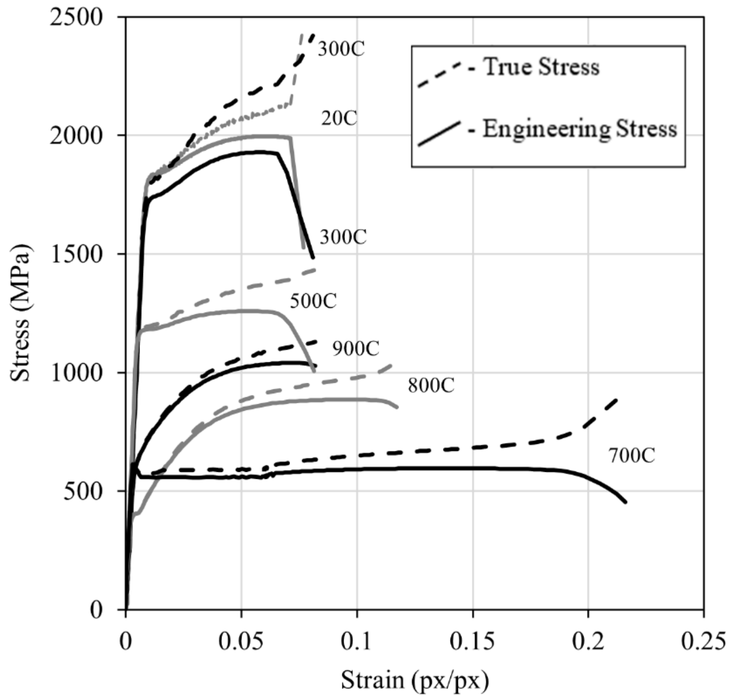

Figure 2. Stress-strain relationship when BS 5896 is heated for two hours.

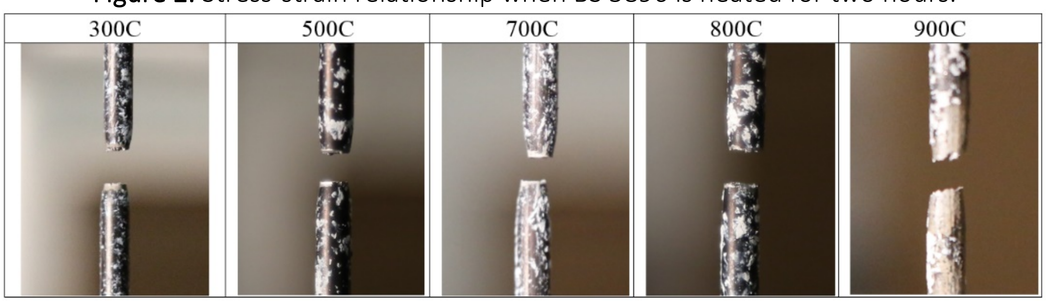

Figure 3. Fracture profiles for BS 5896 after two hours of heating.

The tensile tests displayed that increasing the exposure duration had a significant effect on the residual strength, with a strength reduction of 5 to 10% between a two and eight-hour exposure. The trend indicated that residual strength decreased as heating duration increased, with the exception of BS 5896 when exposed to temperatures higher than 800 °C. In that case, the strength increased with duration, as can be seen in Figure 2 and Figure 4. Figure 4 also shows the repeatability between experiments. The difference between the observed ductility of the samples occurs because of the location of necking; the closer to the grip the necking occurred, the more brittle the fracture appeared. Fracture mechanics suggests that once necking begins, the strain effects become localized.

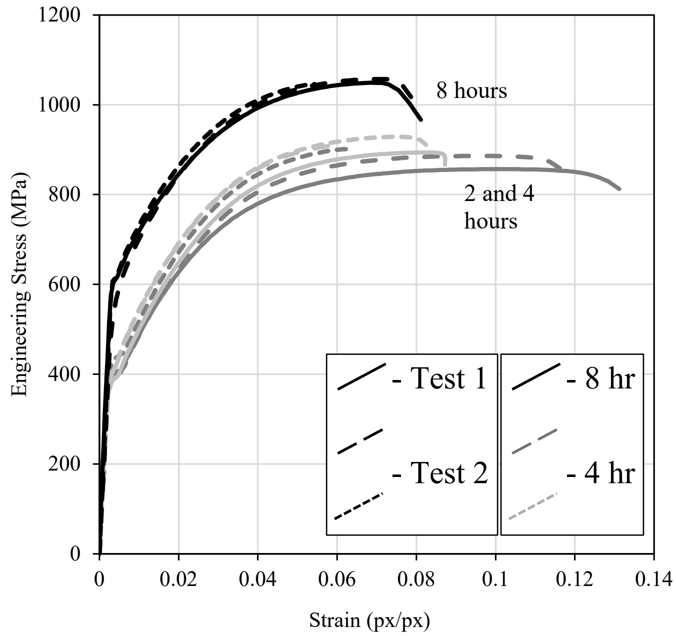

Figure 4. Engineering stress-strain relationship at 800°C for BS 5896 with repeat testing shown.

The gain in strength when exposed to temperatures exceeding 800°C is hypothesized to be caused by a change in grain structure. The manufacturing process of prestressing steel alters the steel grain structure to obtain a higher strength. Exposing the steel to high temperatures undoes this process and returns it to mild steel. This would explain why long exposure durations at low temperatures cause a lower residual strength and why there is a strength gain at 800°C. The AS/NZS 4672 stock of prestressing steel, which has a higher chromium content, was observed to have a lower strength than the stock of BS 5896 steel at temperatures above 500°C. Chromium was thought to help conserve strength when exposed to fires, but these tests question if it has the same pronounced effect after exposure to certain temperatures.

The testing of the prestressing steel has shown that the traditional strength reduction guidance might not be conservative. The guidance predicts that 50% of the strength will remain in the BS 5896 sample after exposure to 600°C for 2 hours, however, the tested sample only had 44% of its original strength. The guidance also does not consider heating for extended durations, which can cause additional strength loss. Before 200°C, the strength of prestressing steel remains approximately the same. Figure 5 illustrates the current reduction factor for both steels, for varying temperatures and exposure durations. Strength reduction factors are calculated by dividing the post high temperature exposure strength by its ambient temperature strength. The strength gain after 800°C can also be seen in Figure 5. The dip in strength of prestressing steel is considered to occur at the Eutectoid temperature, which is when the steel reaches 727°C. The precise transition point where the steel starts to gain strength is unknown and requires more research to be identified. However, this research would not have long term benefits as the critical temperature of prestressing steel occurs before this point.

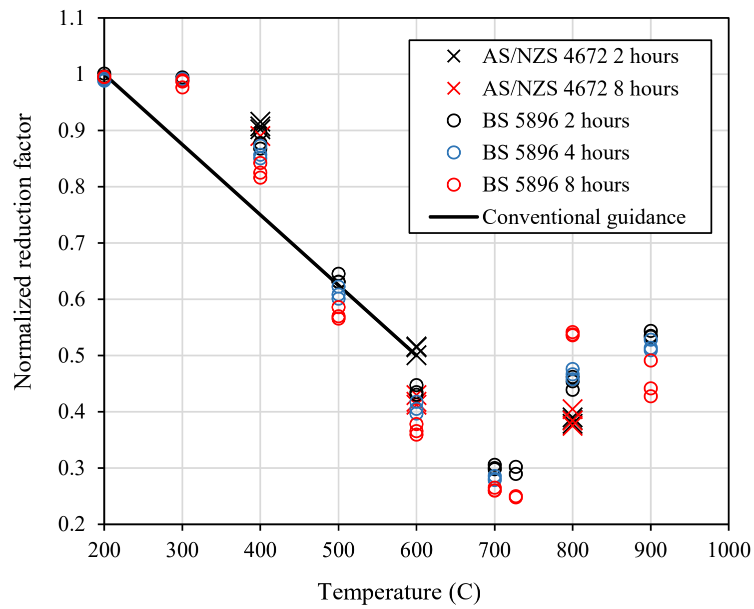

Figure 5. Ultimate strength reduction factors for stocks of BS 5896 and AS/AZS 4672 with conventional guidance after 200°C.

3.2 HARDNESS TESTS

Concurrently with the test program of Section 3.1, small 10 mm samples of prestressing steel were prepared to consider a test plan for the correlation between hardness and strength. Temperatures up to 800°C were considered (re-crystallization occurs after this point and the method herein is unreliable). The microstructure and hardness analyses were performed after samples were correspondingly heated using an annealing furnace following a 10℃/min ramp to peak temperatures (1.5, 4, and 8 hours duration). The microstructure of the prestressing steel was obtained for transverse and longitudinal sections using a Zeiss Axioscope light microscope. Two 10 mm long sections from each sample were cut and mounted in EpoxiCure resin. The samples were grinded using a series of grit papers and then polished with diamond paste and cloths to produce a flat and scratch free surface. To expose the grain structure, the samples were then etched with 2% Nital. The microstructural imaging coupled with the hardness testing was used to identify changes in grain structure. The hardness testing was done using Vickers Hardness tests to obtain a as Diamond Pyramid Hardness (DPH) value. Microstructural images and analysis can be found elsewhere (see companion study from [13]). To avoid false readings from edge effects, the surface of the prestressing steel was grinded to attain a 3 mm wide flat area. Each sample was tested four times and the readings were averaged to obtain a final value. The correlation between strength and hardness is shown in Figure 6. The best fit line can be expressed as the equation below:

$$Ultimate\ strength = 4.336(Hardness\ value\ in\ DPH) - 309.6 \qquad \text{(Eq. 1)}$$

The study showed that hardness testing determines the residual strength of the steel with an accuracy of +/- 10%. It was observed that for temperatures above 600°C, the material properties of the prestressing steel make it more difficult to use the hardness determine the yield strength. The equation remains accurate in relation to the ultimate strength. Therefore, the determined hardness-strength correlation can be used however this method requires destructive analysis of the steel itself which may not be practical in all fire cases.

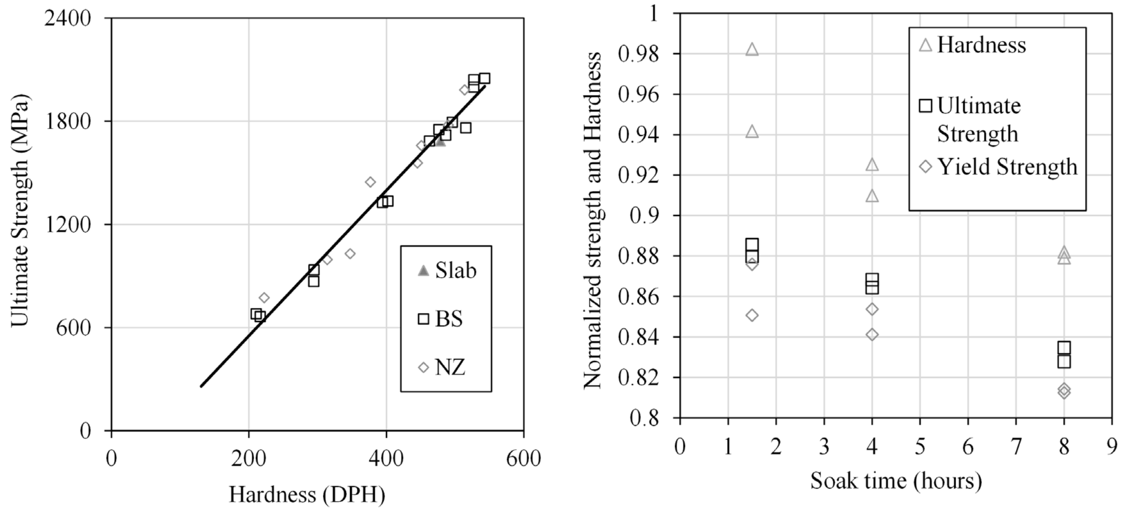

Figure 6. Hardness variation with (left) ultimate strength and (right) soak time at 400°C.

3.3 POST-TENSIONED SLAB TEST AND POST-FIRE INVESTIGATION

A previous test series [29,33,34] included several one-way continuous mono-strand and prestressed concrete slabs (post-tensioned configuration) fixed to four partially-rigid connections and loaded with weights. The tests used the same stock of BS 5896 as those described in Sections 3.1 and 3.2. For verification of the results found in Sections 3.1 and 3.2, one of these tests is analyzed further herein post-fire.

The concrete assembly was heated following a non-standard but quantifiable fire exposure. An important characteristic of the prestressed concrete assembly is that in this case it was unbonded and the steel strand was installed in a sheathed and greased configuration. Different aspects, such as the concrete colour and the recorded temperature exposure of the prestressing steel, can be used to verify the hardness test calibration procedure and recorded strength calibrations for conservatism. It is acknowledged that further studies should be performed to reinforce the conservatism of the presented data in this paper.

Figure 7 shows the time-temperature isotherms of the slab test discussed at the slab's most severely heated location. The slab was exposed to localized heating under mid-span until the prestressing steel reached an in-fire critical temperature of 426 °C (when prestressing steel is said to lose 50% of its strength at elevated temperature). The slab was then allowed to cool to ambient temperature naturally. The tension in the prestressing steel strand was released and a heated section of the prestressing strand was extracted for post strength testing for comparison to Section 3.1 and 3.2's guidance. The concrete in the heated region was also examined for colour changes. The sample of steel was tested for tensile strength, as well as hardness using the method described in the previous section. The DPH value obtained from the hardness testing was converted to a strength value of 1691 MPa using Equation 1. The destructive tensile testing of the central wire of the strand resulted in a stress of 1685 MPa, which represented a 16% drop from its ambient tensile stress capacity of 1950 MPa but only a 9% drop from its grade strength. Since the difference between the strength obtained by hardness testing and by tensile testing is negligible, hardness testing can be assumed to be an accurate method to determine post-fire strength although caution should be used since Equation 1 was derived based upon the values obtained from an exposure time of 1.5 hours. For longer exposures, the residual strength is expected to decrease because of metallurgical changes. This strength reduction effect can be considered to be less than 10% only when the peak temperature does not surpass 400℃, as shown in Figure 6. More research for higher temperatures beyond 426℃ is recommended.

The behaviour shown within Figure 7 is dependent on the depth of the concrete slab and the cover to the reinforcement. The cover thickness impacts the heat transfer process to the steel reinforcement, and ultimately how long and how heated the steel will rise. The reinforcement temperature will also be affected by the surface temperature profile of the slab. The test illustrated within Figure 7 demonstrate only one specific case, and the guidance provided is based upon the peak temperature and its duration.

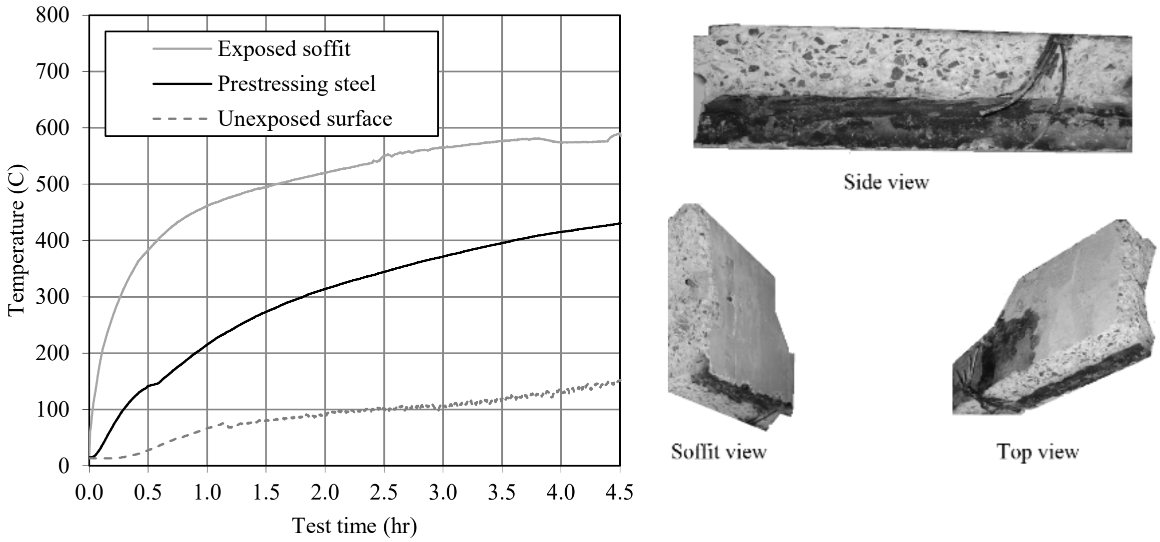

Figure 7. (Left) Measured temperatures for slab test and (right) an excavated slab.

Exposure to fire causes concrete to spall, crack, and change colour depending on the peak temperature. The slabs from the test series had small transverse cracks along their soffits where it had not been exposed to heating. Where the slab had been exposed, longitudinal and transverse cracks were directly below the prestressing steel at the center and either side of the heating zone, respectively. Since concrete changes colour depending on the peak temperature reached, the colour can be used in the post-fire assessment, but only as a general estimation. A 10 mm section of pink concrete from the heated area of the slab was considered. The pink colour observed in the section usually occurs when the concrete temperature exceeds 300°C. The colour alone though is insufficient to identify the actual strand temperature. The black concrete shown in Figure 7 was caused by the melting of the sheathing which seeped out of the concrete during the test.

4. GUIDANCE FOR PRESTRESSED CONCRETE BRIDGES

Determining the post-fire residual strength of prestressing steel using the hardness technique gives results with good accuracy, however this method is only valid up to 700°C. Beyond this point, the microstructure recrystallizes and removes the high strength effects obtained from the manufacturing process. Severe fires where the steel is exposed to fire or where spalling occurs and the prestressing steel becomes exposed should expect the steel to reach temperatures where recrystallization will occur. As shown, the colour change of concrete can be useful to determine the peak temperature reached but it is not always the most reliable. The technique to determine peak temperature from concrete colour must be conservative for it to be used.

In the context of stay-cable members, similar qualitative observations can provide some indication of the maximum temperatures developed. Primarily, the melting of the zinc-aluminum (95% Zn/ 5% Al) galvanization layer that is present on most stay-cable wires is known to melt at approximately 380°C [35] which would be observable on the surface of a cable post-fire. Research examining the effects of air and water-cooling on 20 mm spiral strands heated in a furnace observed, following cooling to ambient, a loss of metallic lustre in galvanized steel cables for temperatures exceeding 300°C and a gradual darkening of the steel with increasing temperature at least to 1000°C which was the maximum temperature studied [36]. Less direct indicators could include the potential liquification or combustion of lubricating blocking compounds in the internal cable structure which, if observed during a fire, could be correlated with the known melting or flashpoint of the substance. Similarly, the potential flashpoint (approximately 330°C) of HDPE cable sheaths used to encase many stay-cables could also be used to approximate temperatures but ultimately may not give sufficient information to determine steel temperatures. Ultimately there has been limited research examining how to assess in-situ stay-cable members following fire exposure and more guidance is needed.

Real fires are highly variable with fluctuating peak temperatures and durations. The traditional strength reduction guidance does not take into account the variability of fires, causing it to be non-applicable when fires have a long duration. Therefore, there is a need for the reduction guidance to be changed in a more conservative manner. The current curves/isotherms being used to describe fires do not account for the variations of fires. For graphs and prescriptive rules to be defensible, they need to consider both the length of exposure (time dependent) and the peak temperature. The difficulty occurs when trying to identify the peak temperature with certainty. The techniques therefore need to be conservative.

Another issue stems from the temperature at which the reduction guidance should be terminated. Traditionally, this occurs at 600°C where the guidance predicts the steel to have reached 50% of its original strength, however, it has been shown that this could occur at as low temperatures as 500°C for long exposure times. The strength gain that occurs at temperatures exceeding 800°C cannot be relied upon when determining the strength reduction factor.

The recommended changes to the guidance are as follows. The decrease in strength will begin at 200°C and will reach 95% of its ultimate strength at 300°C. It will then decrease linearly to 50% at 500°C. These changes are considered valid when compared to the experimental results, even though the change at 300°C reduces the amount of conservatism compared to the traditional guidance. Stopping the guidance at 500°C increases the conservatism. The changes recommended are illustrated in Figure 8.

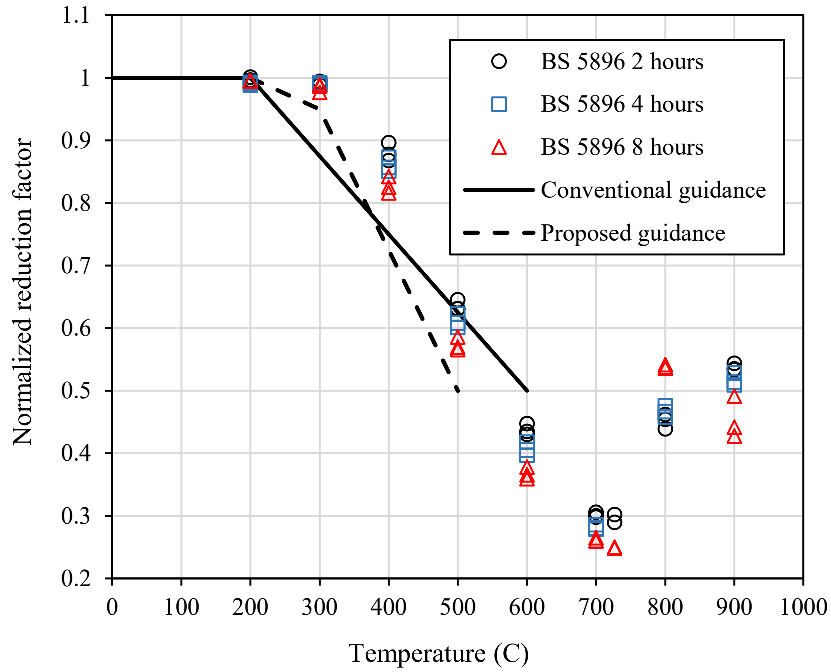

Figure 8. Contemporary prestressing steel strength reduction guidance.

The changes suggested may cause issues with the industry's manufacturers and practitioners, but it is critical to ensure conservative and safe assessment post fire. Regardless of the exposure duration effect, the current reduction factor guidance is not conservative for all temperatures. Determining the prestress remaining within critical infrastructure post-fire is an important task. It needs to be done accurately and efficiently. Two methods discussed to determine the peak temperature reached, concrete colour change and microstructural analysis, are unreliable and time consuming, respectively. The non-destructive test methods need to be improved to be more easily performed and explored further. Measuring the hardness of the prestressing steel is a method to determine the remaining strength of the steel without damaging it, however it cannot be used to determine the duration of exposure or peak temperature reached, which will determine the ductility of the structure.

5. GUIDANCE FOR STAY-CABLE MEMBERS

Cable tension elements used in bridges, stadia, and other critical structures also apply similar high strength carbon steel alloys to those in prestressing steel. Moreover, the vehicle fire exposures expected in bridge structures, for example, can yield high exposure temperatures for extended durations however limited experimental work has examined the fire or post-fire performance of these member types [25]. Therefore, the performance of these steel elements during and following a fire is of interest to researchers and practitioners to aid in the structural fire design and assessment of cable-supported structures. Other fire scenarios, such as the combustion of the HDPE sheaths used to protect stay-cables from corrosion, are credible as well and have resulted in cable losses in real bridge fire events such as in the Chishi (Red-Stone) Bridge in China in 2014. During this event, a total of nine cables were successively lost due to a fire originating in the pylon which ignited the HDPE sheaths and propagated to adjacent cables [27]. The overall fire event lasted just 85 minutes however experimental re-creations of the combustion of the HDPE sheaths indicated flame spread between cables occurred and not all cables were heated simultaneously for the full 85 minutes [37]. Therefore, while dependent on the specific fire scenario, the expected fire exposure time for stay-cable members can be significantly less than that of prestressing steel strands on the basis that stay-cable failure will occur in a relatively short timeframe (unless fire protected which is presently not standard). This implies the two, four, and eight-hour fire exposure times examined in this study will likely overestimate the residual strength of a cable element and future research is needed for this structure type.

A recent experimental test series by the authors [38] have examined the fire performance of unloaded high-strength carbon steel stay-cables during and after exposure to an approximately 30 minute, 0.6 m by 0.49 m methanol pool fire. These experiments measured the temperature and thermal strain development in multiple stay-cables of various diameters and coil configurations during both heating and cooling phases. Optical measurement techniques were applied to record deformations while thermocouples on the surface and embedded in the cables' structure monitored heat transfer to critical regions. Figure 9 demonstrates the experimental configuration of the testing including the cables supported above the pool fire and the observing camera.

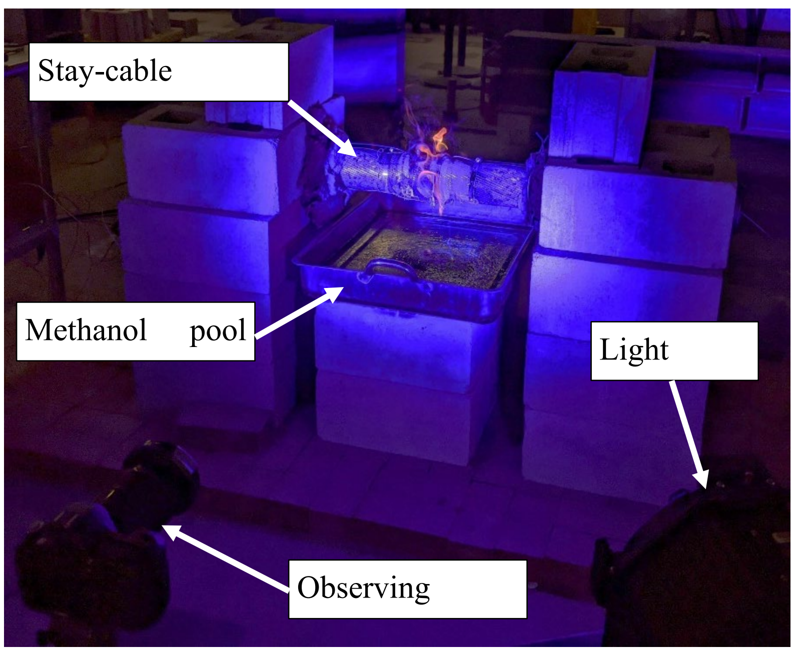

Figure 9. Experimental configuration of the tests conducted by [38]. Shown is a 100 mm locked-coil stay-cable.

The methanol pool fire exposure used in this study produced average gas temperatures at the height of the cables of over 800°C [38]. Similar qualitative indicators of maximum steel temperature as discussed above were observed in this study following the fire, such as the deterioration of the cable galvanized coating corresponding with temperatures of approximately 380°C [38]. Table 3 presents the nominal cable diameter and the maximum temperatures recorded at the core wire and top and soffit surfaces. These tests presented a novel application of high temperature optical measurement technology and were the first experimental study to consider cables of diameter greater than 80 mm. However, this research did not consider the post-fire strength of the cable members based on the maximum recorded temperatures. Therefore, the data herein can be used to compare the residual cable strength range based on the proposed and existing guidance for prestressing steel discussed above. This analysis is presented in Table 3 and applies the measured core wire temperatures for illustrative purposes. Note that, in lieu of specific post-fire heat transfer modelling to determine individual wire temperatures, it would be more conservative to base strength reduction values on the maximum temperature recorded in the cable member and then apply the reduction uniformly for a given cross-section. Herein, the cooler core temperatures are used to compare the proposed and existing guidance since the maximum cable temperatures exceed the relevant temperatures otherwise. As described, although these residual strength values likely overestimate the reduction in strength, this is a useful analysis to conduct as stay-cable elements are highly sensitive to creep in ambient conditions and, for this reason, are often designed to only 50% of their ultimate capacity [39]. Therefore, despite a large reserve in strength in these members, a post-fire reduction in strength may warrant replacement of a cable depending on an enhanced susceptibility to creep in this state. Ultimately, more research is needed to fully understand the post-fire performance of cable elements beyond that of material property reduction.

Table 3. Maximum temperatures and steel strength reduction for each test.

Nominal Diameter (mm)	Steady State Fire Duration (min.)	Soffit Temp. (°C)	Top Surface Temp. (°C)	Core Wire Temp. (°C)	Time to Max. Core Temp. (min.)	Residual Strength Based on Core Wire Temperature	
						Proposed Guidance	Existing Guidance
44	26	776	N/A	612	27	< 0.5	< 0.5
70	34	578	505	488	37	0.53	0.64
74	34	540	462	450	39	0.60	0.69
100	33	501	430	408	37	0.71	0.74
100	32	523	438	405	37	0.71	0.74
140	32	465	338	238	42	0.97	0.95

6. CONCLUSIONS AND FUTURE WORK

With the increasing presence of prestressing steels in critical infrastructure, it is becoming more important to have the tools available to quantify fire damage so that the structures can be assessed and rehabilitated efficiently. The remaining prestress in a structure determines its ability to remain in service and must be determined accurately and with as little intrusion as possible. In lieu of more accurate methods, the guidance currently used in practice must incorporate conservative recommendations that account for the variability associated with fire exposure, especially exposure lengths. Prestressed structures have complex behavior after being exposed to high temperatures. The tests discussed herein address the need for further research upon which new, efficient, and accurate guidance can be based. The current methods are based upon traditional graphical and prescriptive rule guidance and do not account for extended fire exposures which yield higher steel temperatures and greater strength reductions. For this reason, it has been suggested that the strength reduction guidance be updated to account for extended exposures that are likely to occur in real fires. This should be used until a defensible performance-based approach for determining the remaining strength of a post-fire prestressed structures are established. It should be noted that some of the tests described were conducted on prestressing steel that was under no prestress or exterior load. Repeating the experiments in a loaded condition would be the natural next research step for this study and would allow for expanded knowledge of prestressing steel exposed at high temperatures. Further research should also target similar post-fire reduction values for stay-cable members to provide a comparison to the values presented herein. Furthermore, post-fire evaluation tests and methods for cable-supported structures require development, especially in the context of creep susceptibility following a fire. Contemporary technologies like post-tensioned timber for use in bridges are also gaining attention in the industry and will similarly require post-fire guidance and evaluation strategies.

ACKNOWLEDGEMENTS

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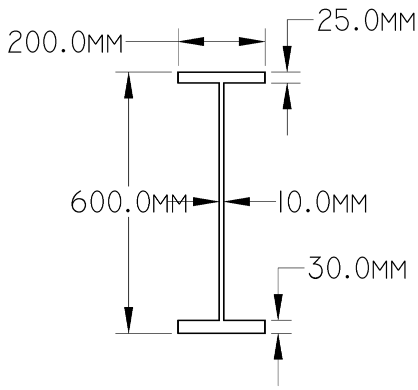
Appendix E:
Scaling Calculations

The scaling calculations were done in such a way to preserve the ratio of the rotational stiffness of the beam to the rotational stiffness of the column. This was done to ensure that the behaviour of the experimental setup as it was heated would remain the same, with a focus on the behaviour of the force transfer through the connection.

<u>Original Dimensions</u>

Beam: PG 670-10-25*200-30*200



Length ~ 16.5 m

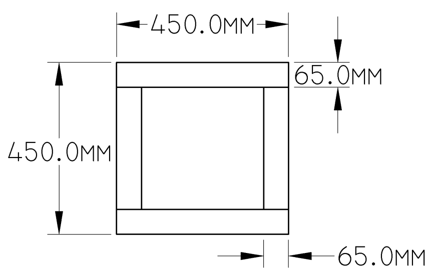
Assume both flanges have thickness of 25 mm for simplicity

Moment of inertia:

$$I = \frac{10 * 620^3}{12} + 2\left(\frac{200 * 25^3}{12} + (200 * 25 * 322.5^2)\right)$$

$$I = 1.239 * 10^9 \ mm^4$$

Column: HK 450-65-65*450-65*450-0



Height ~ 4.0 m

Moment of inertia:

$$I = \frac{450 * 450^3}{12} - \frac{320 * 320^3}{12}$$

$$I = 2.543 * 10^9 \ mm^4$$

Within the original structure, the columns and beams can be assumed to be fixed-fixed. Therefore, the rotational stiffness of all the members can be calculated using the following equation:

$$k = \frac{4 \, E \, I}{L}$$

Where *k* is the rotational stiffness, E is the elastic modulus (MPa), I is the moment of inertia (mm^4) and L is the length (mm).

Since all the members are using this equation for their rotational stiffness and elastic modulus can be assumed to be constant for all steel components, the moment of inertia and the length of each component are the two variables that need to be considered to maintain the same rotational stiffness.

The experimental setup (especially for the future phases of this research which will incorporate hybrid testing) must be scaled to fit within the testing facilities at York University, where the research is conducted.

York High Bay

The strong wall has a height of 6.0 m, as an upper bound of the height of the column.

From iterations, its known that the experimental setup needs to be a quarter-scale from the original dimensions.

The length of the actuator for the hybrid testing is approximately 2.5 m. When the estimated length of the beam (1.5 m) is added, this results in a virtual length of 4.0 m. The strong wall will act as the "centreline" for the beam, therefore the total allowable length of the beam for the scaled calculations is 8.0 m.

To ensure that the experimental setup behaves the same as the original structure, the ratio of the rotational stiffness of the beam and column must remain the same.

$$\frac{k_{beam}}{k_{column}} = \frac{\left(\frac{4EI}{L}\right)_{beam}}{\left(\frac{4EI}{L}\right)_{column}} = \frac{\left(\frac{I}{L}\right)_{beam}}{\left(\frac{I}{L}\right)_{column}}$$

Therefore, the stiffness ratio of the beam to column will stay the same if the ratio of moment inertia (I) to length (L) stay the same for the beam and the column.

Quarter Scale

The lengths of the beam and columns can be scaled by a factor of four, to identify some dimensions at a quarter scale.

Length of original beam = 16.5 m

Length of scaled beam = 16.5 m / 4 = 4.125 m

But since the strong wall acts as the centreline, therefore the beam length for the experiment is half the length of the scaled beam = 2.06 m.

Length of original column = 4.0 m

Length of scaled beam = 4.0 m / 4 = 1.0 m

Scaling down the sections

To scale down the sections, the ratio for the rotational stiffness of the beams to columns is kept constant. Since the properties for the original sections are known, as well as the scaled lengths, the moment of inertia for the scaled sections can be determined.

Beam:

$$\frac{1.239 * 10^9\ mm^4}{16500\ mm} = \frac{I_{scaled,beam}}{4125\ mm}$$

$$I_{scaled,beam} = 309.75 * \ 10^6\ mm^4$$

Column:

$$\frac{2.543 * 10^9\ mm^4}{4000\ mm} = \frac{I_{scaled,column}}{1000\ mm}$$

$$I_{scaled,column} = 635.75 * \ 10^6\ mm^4$$

From the Steel Design Handbook, available steel sections can be identified:

Possible Beams	**I (* 10^6 mm^4)**	**t (mm)**	**w (mm)**
W460x74	332	14.5	9.0
W460x68	297	15.4	9.1
W410x85	315	18.2	10.9
W360x101	301	18.3	10.5
W310x129	308	20.6	13.1

Possible Columns	**I (* 10^6 mm^4)**	**Outside dimension (mm)**	**t (mm)**
HSS 406x406x19	703	406.4	19.05
HSS 406x406x16	606	406.4	15.88

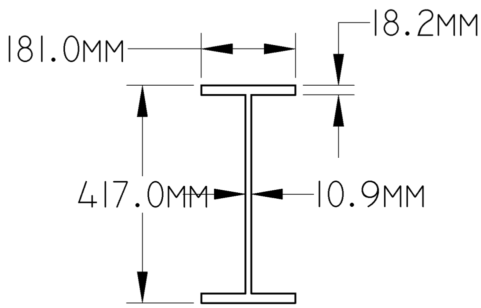
From the available sections that had moment of inertias similar to the calculated values, the beam was selected due to its commonality and weight. The column was selected based on availability. Larger outside dimensions for the column did have moment of inertias within acceptable range, however they were not available in Canada. This column size was the largest available, and therefore the determining factor for the scaling calculations.

Now that sections have been selected, the forces used to design the connections need to be identified.

Connection Forces

Due to the frame having been scaled to ¼ its size based upon its rotational stiffness, the loads upon the system will not be identical to the original design. For this reason, it was decided that using 95% of the ultimate shear and moment resistance would suffice as adequate loads to be applied.

W410x85



A = 10800 mm^2
S_x = 1510 x10^3 mm^3
Z_x = 1720 x10^3 mm^3
h = d – 2t = 417 mm – 2(18.2 mm) = 380.6 mm

Shear resistance (CSA S16-19 clause 13.4.1.1)

$$V_r = \varphi A_w F_s$$

Where A_w = 2ht = 2(380.6 mm)(18.2 mm) = 13 853.84 mm^2

Assuming unstiffened webs: h/w = 380.6 mm / 10.9 mm = 34.9 ≤ $\frac{1014}{\sqrt{350\ MPa}}$ = 54.2, therefore use

$$F_s = 0.66\ F_y = 0.66\ (350\ \text{MPa}) = 231\ \text{MPa}$$

Therefore,

$$V_r = \varphi A_w F_s$$

$$V_r = 0.9 * 13853.84\ mm^2 * 231\ MPa$$

$$V_r = 2\ 880\ 213.3\ N = 2880\ kN$$

Bending resistance (CSA S16-19 clause 13.5)

Assuming that the beam is laterally supported and exceeding class 1 & 2:

$$M_r = \varphi Z_x F_y = \varphi M_p$$

$$M_r = \varphi * (1720 * 10^3\ mm^3) * 350\ MPa$$

$$M_r = 541.8\ kNm$$

95% capacity to calculate maximum load

$V_{r,95\%}$ = 0.95(2880.2 kN) = 2736.2 kN

$M_{r,95\%}$ = 0.95(541.8 kNm) = 514.7 kNm

Now that the shear and moment capacities of the beam are known, there is a need to determine which will govern the failure of the beam. In the original structure, the beam can be assumed to be uniformly loaded. Therefore, to determine whether shear or moment will govern the loading, both cases will be examined. The case with the smallest distributed load to failure will govern.

Shear capacity

$$V_a = \frac{w_a\, l}{2}$$

$$w_a = \frac{2\, V_a}{l} = \frac{2\,(2736.2 * 10^3\, N)}{(4125\, mm)} = 1326.6\, kN/m$$

Moment capacity

Assuming that the beam is fixed-fixed:

$$M_a = \frac{w_a\, l^2}{8}$$

$$w_a = \frac{8\, M_a}{l^2} = \frac{8\,(514.7 * \ 10^6\, Nmm)}{(4125\, mm)^2} = \ 241.99\, kN/m$$

The smaller distributed load would govern, therefore w_a = 242 kN/m and the moment capacity governs.

The force on the column is the support reaction of beam:

$$V_a = \frac{(4.125\, m)\,(242 \frac{kN}{m})}{2} = 499.1\, kN$$

The maximum eccentricity is typically 125 mm from face of supporting member. The force should be applied at the centre of the beam. Therefore,

e_{max} = 177.8 mm + 125 mm = 302.8 mm

Therefore, the connection should be designed for V_a = 499 kN with e_{max} = 302.8 mm.

Appendix F:
Shear Tab Connection Design

Provided by:

Benson Steel Ltd.

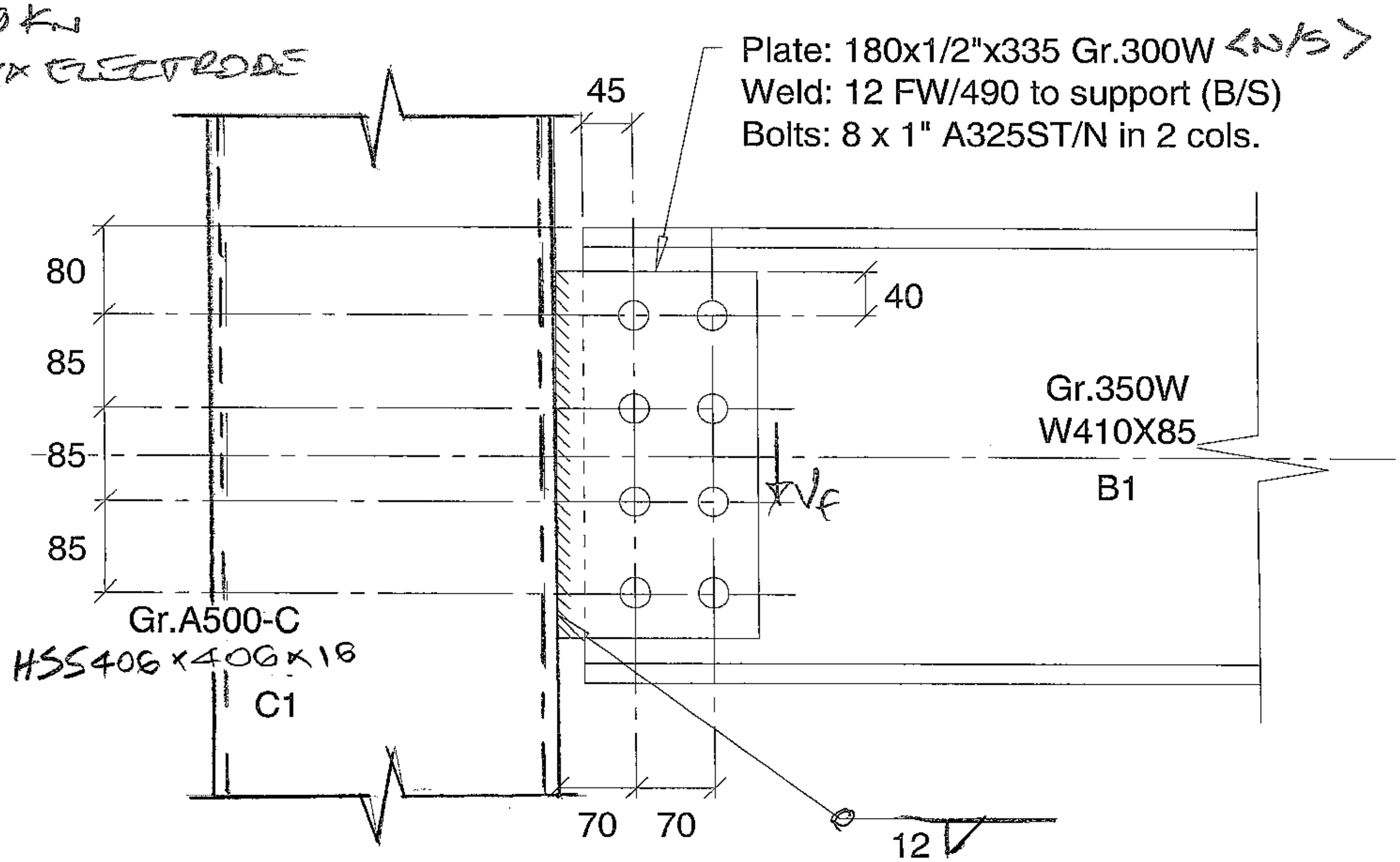
BENSON STEEL

08/07/2020
09:36:19 AM

Boro
Job: LimconJob

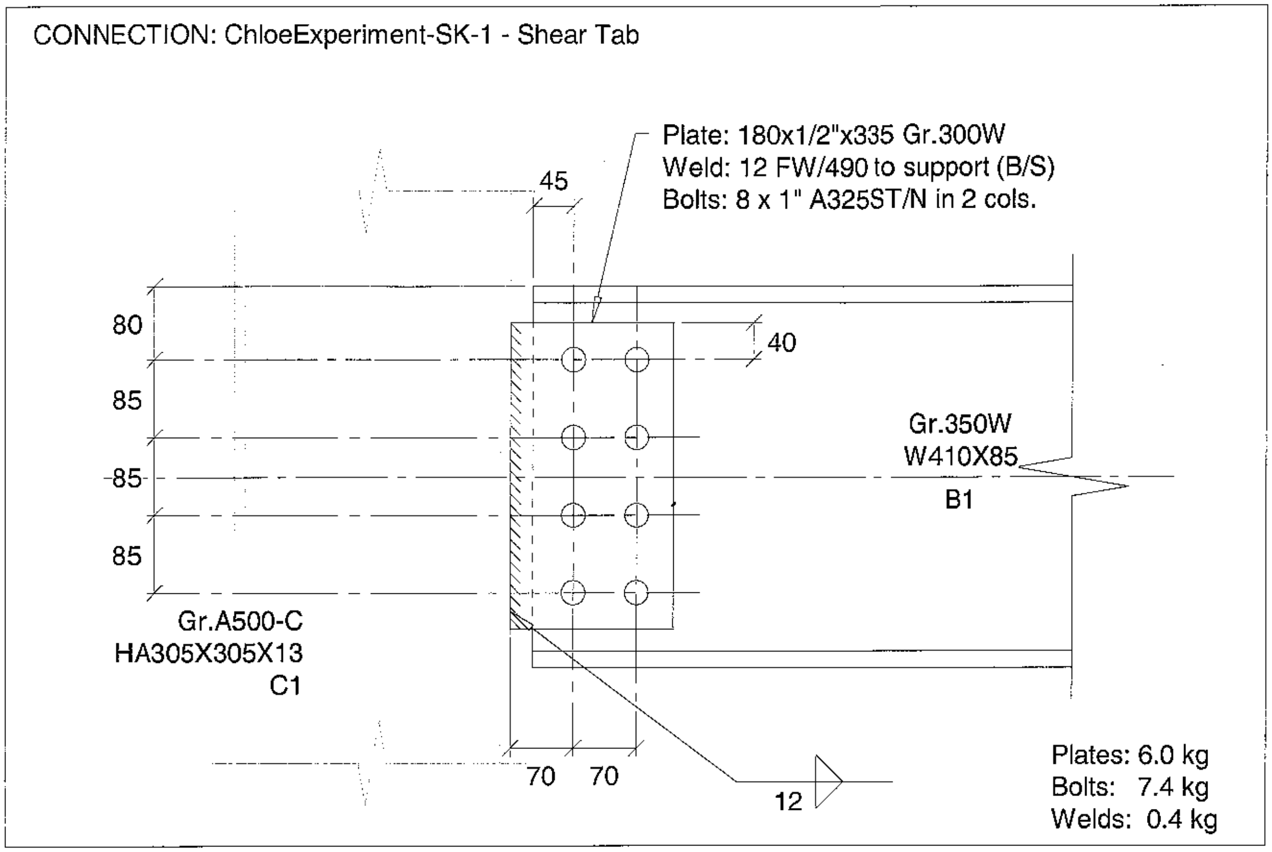
CONNECTION: ChloeExperiment-SK-1 - Shear Tab

V_f = 499 kN
<E49XX ELECTRODE>



Plates: 6.0 kg
Bolts: 7.4 kg
Welds: 0.4 kg

Boro
Job: LimconJob -- Connection: ChloeExperiment-SK-1




```
 |Edge distance dia. multiple . .   1.500
 |Min. spacing dia. multiple  . .   2.700
 |Min. spacing dia. multiple  . .   2.700
LIMCON V3.63.2.6 {0}                                                        08-JUL-20
                                                                             09:35:13

 Connection: ChloeExperiment-SK-1
       Type: Shear Tab
    Country: Canada
      Units: SI metric
Design code: CSA-S16

Beam:  Mark=B1  Section=W410X85   Grade=350W   Span=1.7 m
     d =    417 mm   Root rad. =     15 mm    Fyf =     350 MPa
     b =    181 mm        Area = 1.0800E+04   Fyw =     350 MPa
    tf =   18.2 mm          Sx = 1510000      Fu =      450 MPa
    tw =   10.9 mm          Zx = 1720000
   |Aw  . . . . . . . . . . . . . . .    4545 mm2
   |Fy  . . . . . . . . . . . . . . .     350 MPa
   |h/w . . . . . . . . . . . . . . .   34.92
   |kv  . . . . . . . . . . . . . . .    0.00
   |Fs  . . . . . . . . . . . . . . .     231 MPa
   |ø . . . . . . . . . . . . . . . .    0.90
 .Section shear strength . . . . .    945.0 kN                       S16-13.4.1.1a1
 .Section tension strength . . . .   3402.0 kN                       S16-13.2(a)(1)
 .Section compression strength . .   3402.0 kN                       Note 4

Plate:
 335x180x13  Gr./Fy/Fu=300W/30/450MPa

Bolts:
 8 x 1" A325ST/N in 2 columns.
 Bolt pitch, sp  . . . . . . . . .      85 mm
```

```
 Support to 1st bolt column, sg1 .      70 mm
 Bolt gauge, sg2 . . . . . . . . .      70 mm

Weld:
 12 FW/490MPa to support.

Support:  Mark=C1  Section=HA305X305X13   Grade=A500-C
     d =    305 mm        Area = 1.3100E+04     Fy =     345 MPa
     b =    305 mm          Sx = 1210000        Fu =     427 MPa
     t =   11.4 mm          Zx = 1430000
 .EC3 section class  . . . . . . .        1
 .Section compression strength . .  4064.5 kN                              Note 4
 Unspecified support condition.

BILL OF MATERIALS
Plates:
 1 no. - 335x180x13 Grade=300W . . . . . . .    6.0 kg
Bolts:
 8 no. - 1" A325ST/N x 80 long . . . . . . .    7.4 kg
Welds:
 670 mm - FW 12 fu=490MPa  . . . . . . . . .    0.4 kg

MINIMUM SHEAR FORCE CHECK
 Input design actions are not automatically increased if they are less than the specified
 minimum actions. Minimum actions may be set in any load case. This check warns if the
 shear force is less than the specified minimum (40 kN) for all load cases.
 * Shear force exceeds specified minimum in at least one load case.

INPUT DESIGN ACTIONS
 Shear, Vu . . . . . . . . . . . . .  499.0 kN
 Axial, Pu . . . . . . . . . . . . .    0.0 kN
  |ø . . . . . . . . . . . . . . . .   0.80
  |Bolt Fu . . . . . . . . . . . . .    825 MPa

Bolt group eccentricity  . . . . .  105.0 mm
Max. eccentricity moment . . . . .   52.4 kN.m

BEAM TABLE SHEAR
 For uniform load on simply supported beam:
  Section moment strength  . . . .  541.8 kN.m                             S16-13.5a
  |Aw  . . . . . . . . . . . . . .   4545 mm2
  |Fy  . . . . . . . . . . . . . .    350 MPa
  |h/w . . . . . . . . . . . . . .  34.92
  |kv  . . . . . . . . . . . . . .   0.00
  |Fs  . . . . . . . . . . . . . .    231 MPa
  |ø . . . . . . . . . . . . . . .   0.90
  Section shear strength . . . . .  945.0 kN                               S16-13.4.1.1a1
  Span . . . . . . . . . . . . . .  1.7 m
  Max. total uniform load  . . . . 1889.9 kN
  Shear  . . . . . . . . . . . . .  945.0 kN

Using AISC SCM 14th model...

GEOMETRY CHECKS
CHECK 1 - Detailing Requirements:
  |Edge distance dia. multiple . .  1.500
 Ref. 8: Handbook of Steel Construction - 8th Edition - CISC - 2004 (HSC8)
 == This is an EXTENDED CONFIGURATION connection. ==
 Plate fillet weld leg . . . . . .    12.0          ≥      9.5          Yes
 Plate length  . . . . . . . . . .     335          ≥      175          Yes    SCM13 p.10-104
 Plate Fy  . . . . . . . . . . . .     300          ≤      310          Yes
 Plate thickness . . . . . . . . .    12.7          ≥      6.0          Yes
                                      12.7          ≤     14.7          Yes
 Plate vertical edge distance  . .      40          ≥       38          Yes
 Plate horizontal edge distance  .      40          ≥       38          Yes
 Web horizontal edge distance  . .      45          ≥       38          Yes

DESIGN CAPACITY CHECKS
                                        Strength ratio
                                        Required strength
                                        Design strength                        Reference
```

Boro
Job: LimconJob -- Connection: ChloeExperiment-SK-1

```
CHECK 2 - Weld:
 Weld length (each side) . . . . .    335 mm
  |Weld leg  . . . . . . . . . . .     12 mm
  |Weld metal Fu . . . . . . . . .    490 MPa
  |Weld metal strength . . . . . .  1.866 kN/mm                                   S16-13.13.2.2
  |Base metal Fu . . . . . . . . .    427 MPa
  |Base metal shear rupture  . . .  2.303 kN/mm                                   S16-13.13.2.2
  |»Effective weld strength  . . .  1.866 kN/mm
 Weld design moment  . . . . . . .   52.4 kN.m
 Weld strength (elastic) . . . . . . . . . .  587.1  ≥  Vu   =  499.0  1.18  Pass

CHECK 3 - Bolts:
 Bolt group: 8 x 1" A325ST/N
 Bolt group design eccentricity  .  105.0 mm
 Bolt group design moment  . . . .   52.4 kN.m
  |ø . . . . . . . . . . . . . . .   0.80
  |Bolt Fu . . . . . . . . . . . .    825 MPa
 Single bolt shear strength  . . .  140.5 kN                                      S16-13.12.1.2c
 Zb  . . . . . . . . . . . . . . .  4.247
 Zev . . . . . . . . . . . . . . .  0.736
 Zeh . . . . . . . . . . . . . . .  0.766
 Bolt group shear strength . . . . . . . . .  596.5  ≥  Vu   =  499.0  1.20  Pass
  |Fy  . . . . . . . . . . . . . .    300 MPa
  |Fu  . . . . . . . . . . . . . .    450 MPa
  |ø . . . . . . . . . . . . . . .   0.80
  |Single bolt bearing strength  .  348.4 kN                                      S16-13.12.1.2a
 Plate bearing strength  . . . . . . . . . . 1479.5  ≥  Vu   =  499.0  2.96  Pass  S16-13.12.1.2a
  |Fy  . . . . . . . . . . . . . .    350 MPa
  |Fu  . . . . . . . . . . . . . .    450 MPa
  |ø . . . . . . . . . . . . . . .   0.80
  |Single bolt bearing strength  .  299.0 kN                                      S16-13.12.1.2a
 Web bearing strength  . . . . . . . . . . . 1269.8  ≥  Vu   =  499.0  2.54  Pass  S16-13.12.1.2a
  |Dh  . . . . . . . . . . . . . .   27.0 mm
  |Agv . . . . . . . . . . . . . .   1016 mm2
  |Fy  . . . . . . . . . . . . . .    300 MPa
  |Fu  . . . . . . . . . . . . . .    450 MPa
  |Feff  . . . . . . . . . . . . .    375 MPa
  |ø . . . . . . . . . . . . . . .   0.75
  |Single bolt tearing strength  .  171.5 kN                                      S16-13.11
 Plate vert. tearing (top) . . . . . . . . . 1009.8  ≥  Vu   =  499.0  2.02  Pass  S16-13.11
  |Dh  . . . . . . . . . . . . . .   27.0 mm
  |Agv . . . . . . . . . . . . . .   1559 mm2
  |Fy  . . . . . . . . . . . . . .    350 MPa
  |Fu  . . . . . . . . . . . . . .    450 MPa
  |Feff  . . . . . . . . . . . . .    400 MPa
  |ø . . . . . . . . . . . . . . .   0.75
  |Single bolt tearing strength  .  280.6 kN                                      S16-13.11
 Web vert. tearing (int.)  . . . . . . . . . 1652.6  ≥  Vu   =  499.0  3.31  Pass  S16-13.11

CHECK 4 - Plate:
  |Ag  . . . . . . . . . . . . . .   4255 mm2
  |An  . . . . . . . . . . . . . .   2884 mm2
 Plate shear yield strength  . . . . . . . .  689.2  ≥  Vu   =  499.0  1.38  Pass
 Plate shear rupture strength  . . . . . . .  583.9  ≥  Vu   =  499.0  1.17  Pass  S16-13.11
  |Plate plastic modulus . . . . . 356314 mm3
  |Plate yield moment, øMn . . . .   96.2 kN.m
 Flexural yield strength . . . . . . . . . .  916.2  ≥  Vu   =  499.0  1.84  Pass  S16-13.5a
  |Net plastic modulus . . . . . . 239782 mm3
  |Plate rupture moment, øMn . . .   82.5 kN.m
 Flexural rupture strength . . . . . . . . .  786.1  ≥  Vu   =  499.0  1.58  Pass  SCM13 II.A-19
  |Nominal plate shear stress  . .    117 MPa
  |øFcr (von Mises)  . . . . . . .    199 MPa
  |Elastic modulus . . . . . . . . 237542 mm3
  |Von Mises moment strength . . .   47.2 kN.m                                     SCM13 p.10-103
  |(Vr/Vc)2 + (Mr/Mc)2 . . . . . .   0.82                                          SCM14 (10-5)
  |Reduced moment strength . . . .   66.4 kN.m                                     SCM14 p.10-104
 Flexural strength reduced for shear . . . .  632.0  ≥  Vu   =  499.0  1.27  Pass  SCM14 p.10-104
  |Agv . . . . . . . . . . . . . .   3747 mm2
  |Anv . . . . . . . . . . . . . .   2547 mm2
  |Ant . . . . . . . . . . . . . .    883 mm2
  |Fy  . . . . . . . . . . . . . .    300 MPa
  |Fu  . . . . . . . . . . . . . .    450 MPa
```

Boro
Job: LimconJob -- Connection: ChloeExperiment-SK-1

```
  |Feff . . . . . . . . . . . . . .     375 MPa
 Block shear strength . . . . . . . . . . .   721.6  ≥  Vu    =  499.0  1.45  Pass  S16-13.11

CHECK 5 - Beam Shear:
  |Aw . . . . . . . . . . . . . . .     4545 mm2
  |Fy . . . . . . . . . . . . . . .      350 MPa
  |h/w . . . . . . . . . . . . . .    34.92
  |kv . . . . . . . . . . . . . . .     0.00
  |Fs . . . . . . . . . . . . . . .      231 MPa
  |ø . . . . . . . . . . . . . . .      0.90
 Shear strength . . . . . . . . . . . . . .   945.0  ≥  Vu    =  499.0  1.89  Pass  S16-13.4.1.1a1

CHECK 6 - Coped Beam Block Shear:
 Beam not coped » check not required.

CHECK 7 - Coped Beam Bending:
  |e . . . . . . . . . . . . . . .      110 mm
 Beam not coped » check not required.

CHECK 8 - Beam Rotation:
  |Span . . . . . . . . . . . . . .  1.7 m
 Rotation for UDL (rad.) . . . . .    0.002
 Rotation for contact (rad.) . . .    0.119
 Contact rotation strength . . . . . . . . .  31216  ≥  Vu    =  499.0  62.6  Pass
 Max. bolt shear deformation . . .      0.3 mm
 Max. recommended deformation  . .      6.0 mm
 Strength at bolt deformation limit . . . .   11871  ≥  Vu    =  499.0  23.8  Pass  Informative

CHECK 9 - Coped Beam Buckling:
 Beam not coped » check not required.

CHECK 10 - Support Member:
  |Fy . . . . . . . . . . . . . . .      345 MPa
  |Fu . . . . . . . . . . . . . . .      427 MPa
  |ø . . . . . . . . . . . . . . .      0.90
 HSS column wall shear yield strength . . .  1568.2  ≥  Vu    =  499.0  3.14  Pass
 HSS column wall shear rupture strength  . .  1652.9  ≥  Vu    =  499.0  3.31  Pass
 HSS shear rupture capacity at welds...
  |Throat . . . . . . . . . . . .      8.5 mm
  |Weld metal Fu . . . . . . . . .      490 MPa
  |ø . . . . . . . . . . . . . . .      0.67
  |Weld metal strength . . . . . .    1.866 kN/mm
  Fillet weld strength . . . . . .    1.866 kN/mm
  |Ag . . . . . . . . . . . . . .       11 mm2
  |Fu . . . . . . . . . . . . . .      427 MPa
  HSS wall shear rupture strength     2.199 kN/mm
  Min. permitted wall thickness  .      9.7 mm
  HSS wall thickness . . . . . . .     11.4             ≥     9.7                  Yes   SCM14 p.9-5
 Punching shear...
  HSS wall thickness . . . . . . .       11 mm
  |twc . . . . . . . . . . . . . .     11.4 mm
  |fuc . . . . . . . . . . . . . .      427 MPa
  |fyi . . . . . . . . . . . . . .      300 MPa
  Max. permitted plate thickness .     16.3 mm
  Plate thickness . . . . . . . .      12.7             ≤    16.3                  Yes   A360 (K1-10)

NOTES:
  4. Compression yield capacity of section ignoring slenderness.

CRITICAL LIMIT STATE . . .  Plate shear rupture strength
UTILIZATION RATIO . . . .     85%
STRENGTH RATIO . . . . . .   1.170   Pass
```

Appendix G:
Human Behaviour in Informal Settlement Fires in Costa Rica

From:

Arces, S., Jeanneret, C., Gales, J., Antonellis, D., and Vaiciulyte, S. (2021) Human Behaviour in Informal Settlement Fires in Costa Rica. Safety Science (Elsevier) 142

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ABSTRACT

Globally, 1 billion people live in informal settlements, which are typically highly susceptible to fire. Small fires can quickly evolve into large conflagrations causing significant losses of life and property, injuries, and subsequent exacerbation of residents' existing vulnerabilities. Informal settlement fires are a complex socio-technical phenomenon which involve a combination of fire dynamics, structural and behavioural response. These fires are not broadly researched nor are they well understood. In this paper, the authors attempt to illustrate this multi-faceted issue and provide recommendations to help guide future research.

Emphasis is placed on human behaviour in response to fires, including fire response activities by residents and city fire brigades. The case study presented in this paper is a 2019 fire which occurred in El Pochote, an informal settlement located in San José, the capital of Costa Rica. Through the Costa Rican Fire Corps, video footage recorded live and in-situ (70 minutes) of the brigades' and residents' actions in this event was obtained. This video shows real-time human behaviour and fire response activities, as well as complex fire dynamics and resulting structural reactions. It is clear from this footage that human behaviour responses to informal settlement fires are highly coupled with local context and specific situations, demonstrating limitations with extending observations from a specific case study to informal settlements more broadly.

This paper introduces a methodology for situational analysis and documentation of human behaviour responses based on video footage of informal settlement fires. It provides key insights to how key stakeholders such as the Costa Rican Fire Corps, the police and informal settlement dwellers interact with each other and respond to fires in informal settlements in Costa Rica. These results form a first-stage contextualization of informal settlement fires in Costa Rica, which can be used to inform the Costa Rican Fire Corps, non-governmental organizations and other stakeholders that may be able to support fire safety improvements in Costa Rican informal settlements. Despite the challenging nature of studying human behaviour from real-life footage, the authors aim to establish a foundation for future research in compiling evidence of human behaviour in informal settlement fires across different countries.

1. INTRODUCTION AND MOTIVATION

There is currently a global trend of population growth, and it is expected that the world population will rise 13% by 2050 [1]. This growth will be unequally distributed geographically, driving a continuous and faster urbanization in low to middle income countries [1]. This can be observed in San José, Costa Rica, where the employment opportunities presented to the population in the Great Metropolitan Area are greater than in the countryside. This is leading to the migration of the population from rural to urban areas. However, a lack of affordable housing in cities has resulted in the creation of new informal settlements in and around San José and increasing densities of existing informal settlements in the area.

UN-Habitat defines 'informal settlements' as "residential areas where inhabitants have:

- No security of tenure vis-à-vis the land or dwellings they inhabit;
- Neighbourhoods usually lack, or are cut off from, basic services and city infrastructure;
- Housing may not comply with current planning and building regulations, and is often situated in geographically and environmentally hazardous areas." [2]

Considering the extensive degrees of spatial, social, economic, and political marginalization people living in informal settlements often experience, it is critical to acknowledge that 'informal' does not mean illegitimate or temporary. The lack of formality associated with informal housing is generally the result of historic prejudices or conflicts, governance failures, and failures in socioeconomic and political systems. Regardless of the root causes, people living in informal settlements bear the burden, often with increased exposure to hazards and substantial vulnerabilities – potentially leading to additional consequences. Poor people are nearly twice as likely to live in fragile dwellings, and while not all people living in informal settlements are poor, there are positive correlations between poverty and informality [3]. Spinardi et al. [4] stated that it is widely believed that the fire problem is greater in the poorer economies of the world, and particularly in informal settlements, and better data collection and statistical analysis would be an important step towards addressing this problem.

Fire risk in informal settlements is a function of complex interactions between the built environment, the natural environment, and people. The high density of dwellings is one factor. The use of combustible construction materials, like timber, plastic, cardboard, and steel sheets, in close proximity is another. Open cooking, issues with lighting and poor electrical connections present ignition sources. Research from South Africa found flashover inside local informal settlement dwellings can happen in less than a minute, and there's little time before it spreads to adjacent dwellings [5]. Fires in informal settlements often spread between dwellings and can quickly evolve into large conflagrations, influenced by wind and other environmental factors. Fire services response to informal settlement fires is often hampered by delayed notification, challenges finding the location of the fire, poor access due to a lack of road infrastructure and low hanging electrical wires, limited water infrastructure to support firefighting efforts, and in some cases, social tension with residents, where the fire services response is perceived to be too slow by residents, for example.

Many people living in informal settlements incrementally upgrade and expand their dwellings over time. In Costa Rica, some houses are several stories high, which can increase fire risks when compared to the single-story houses due to factors such as increased ignition risks (linked with population density), higher fuel load densities, and higher likelihood and consequences of structural collapse during a fire. Multi-story dwellings also pose direct risks to life, due to potential delays or complications with evacuation or search and rescue activities. Figure 1 depicts the construction of houses with several stories in La Carpio, an informal settlement in San José.

Figure 1. Multi-story houses in La Carpio (source: author's photo. May 7th, 2019)

According to the census carried out in 2011 by the Costa Rican National Institute of Statistics and Censuses, there are around 418 informal settlements in the country, which house more than 220,000 inhabitants. Costa Rica has a population of 5 million at the time of writing, meaning more than 4 percent of the country's population live in informal settlements. The metropolitan city of San José hosts approximately 104 informal settlements, making it the city with the most informal settlements in the country [6].

According to fire reports from the Engineering Unit of the Costa Rican Fire Corps, the number of fires in Costa Rican informal settlements has been fluctuating over the last five years [7], though information is sparse to the accuracy of this reporting (fires may be underreported or handled internally without reporting). In 2018, the Engineering Unit reported a total of 1126 structural fires, of which 50 fires occurred in informal settlements [7] [8]. Within all of the structural fires, it was reported that a total of 30 people perished due to fire-related incidents [8], which is the highest number observed in the previous nine years [9].

In 2019, two large-scale fire tragedies occurred in informal settlements in San José. The first fire occurred in a leased dwelling located inside the informal settlement called La Carpio on April 13th. Seven people died in this isolated house fire [10]. Figure 2 is an image of this dwelling after the fire, which was constructed of masonry and concrete, and bound on all sides. The other fire occurred on September 16th in El Pochote, an informal settlement located in the Barrio Cuba region. This fire spread through forty dwellings and left 219 people homeless [11]. Video footage of the development and responses to this fire, as well as post-fire damage, was obtained by the authors for analysis through the Costa Rican Fire Corps.

Figure 2. Post-fire in La Carpio, Costa Rica (source: authors' photo. May 7th, 2019)

Over the past 5 years, there has been an emerging field of research into fires in informal settlements. While this is not a new problem or the first research in this area, recent efforts have been at a much larger scale and with stronger dissemination and communications than previous decentralized efforts. Globally, there is growing recognition of the importance of addressing fire challenges in these contexts. For example, Arup developed a framework that proposes a holistic sociotechnical approach to fire safety in informal settlements [12]. Referencing the widely used disaster cycle, the framework aims to establish an accessible common language to promote collaboration among diverse stakeholders. It provides a conceptual framing for fire safety challenges and opportunities in these complex environments and introduces a resilience-based approach to fire safety. The framework specifically addresses cultural differences and beliefs that may present challenges in considering fire safety or evacuation strategies [12].

Another example is a research project known as IRIS-FIRE, developed by engineers and social scientists, which is looking to improve the resilience of informal settlements to fire [13]. Conducted by the University of Edinburgh (United Kingdom) and Stellenbosch University (South Africa), IRIS-FIRE focuses on informal settlements primarily located in the Western Cape province of South

Africa [13]. At the time of writing, a literature review of the work completed by IRIS-FIRE shows their research efforts were concentrated on two categories: technology/modelling and full-scale experiments for understanding fire dynamics, structural and material response. For example, Gibson et al. [14] published a paper which outlines a new approach to mapping historic and ongoing fires in informal settlements using Sentinel satellite imagery. Others focused on modelling fires and predicting their spread in informal settlements by using B-RISK technology [15] and Fire Dynamics Simulator [16]. IRIS-FIRE conducted several outdoor full-scale fire tests on representative informal settlement dwellings to analyse compartment fire behaviour and fire spread between dwellings. Informal settlement fires in the Western Cape often spread between dwellings and through communities due to high densities and proximities of dwellings [17]. Factors such as heat release rates (HRR), heat fluxes, time of ignition, and the effect of fuel load were studied through experimentation. Based on this research, methods have been proposed to limit fire spread and reduce fire risk generally. Recently, these researchers have begun collating videos of real fires in South African settlements and are beginning to consider human behaviour [18].

Even though there has been some seminal research, there are still significant gaps in informal settlement fire safety research. Scientific research of human behavioural responses to fires in informal settlements is severely lacking. There is no study, to the authors' awareness, which links current behavioural theories to informal settlement fire scenarios. This is despite recent advancements in the development of modern behavioural theory frameworks for fire generally.

For example, Kinsey et al. [19] have studied the role of cognitive biases in behaviour during fires and provided short statements to help practitioners contextualize a person's decision-making process during a fire. Cognitive biases, however, have been associated with inaccurate judgements, as shortcuts used for decision-making can sometimes lead to important information or actions being overlooked [20]. Another theory is the Protective Action Decision Model (PADM) [21]. PADM is a provisional multistage decision-making model based on the research of people's responses to environmental hazards and disasters – it attempts to describe the pre-decisional and decisional sequence of action making. PADM integrates the processing of information derived from social and environmental cues with the messages transmitted through communication channels [21]. In previous research, PADM has been adapted to households' response to bushfire threat. For instance, research conducted by Folk et al. [22] aimed to create a survey to collect further information about the factors influencing protective action decision making during wildfires. As well, Strahan and Watson's study addresses the question of how PADM can be applied to the analysis of behaviour in Australian bushfires and North American wildfires [23]. More recently in the context of mass gatherings, social identity and influence theories have also been proposed to help describe decision making within a group [24]. That research implies that during a fire or emergency, a collective (shared) approach to decision making can occur, though this is still emerging with more research attention in the context of evacuation or fighting of fires. These frameworks are only a few of the many behavioural frameworks currently being developed and explored by researchers in fire safety.

While these behavioural theories provide insights to the way people process information in emergency scenarios in general, their relevance to the way people behave during fires in informal settlements is not yet known. Applying these theories (and others) in this study could generate inaccurate outcomes since there is insufficient data for validation, resulting in bounding the behaviours observed in frameworks which may not necessarily be appropriate. Empirical evidence from several informal settlement fires in similar contexts is needed to enable observations of common behaviours and to better understand the interactions between people, their environment, and fire. Current knowledge gaps around fire behaviour in informal settlements compounds uncertainties associated with linking human responses to environmental cues for example. While it is premature to apply existing behavioural theories to informal settlement fires, and it is possible entirely new frameworks may need to be developed, studies as herein can contribute behavioural observations from real fire incidents.

Using the obtained video footage of an informal settlement fire, this first stage research attempts to consider four principal goals that begin to address the identified research needs: (1) to collect human behaviour and fire evolution data in an informal settlement, (2) to illustrate some of the key variables that make informal settlements fires difficult to study, (3) to consider the challenges faced when extending observations beyond a specific fire, and (4) to propose a methodology for collection of behavioural data in informal settlements for post fire analysis of other informal settlement fires in the future. By compiling a body of evidence of human and fire behaviour in an informal settlement fire, this study aims to support future research efforts and to enable the eventual development of a fire response framework for informal settlements in Costa Rica.

2.0 FIREFIGHTING AND EMERGENCY RESPONSE PROCEDURES

2.1 METHODOLOGY

To holistically understand informal settlement fires, it is important to seek the perspectives of key stakeholders, namely residents, firefighters, and others who support communities in times of crises such as during or after a fire. Due to ethical concerns, it was not possible for the authors to engage directly with the residents involved in informal settlement fires (further details in section 3.0), but the authors were able to engage with firefighters in San José, who offered valuable insights about fire responses in informal settlements.

Through this project, two sessions of informal and semi-structured oral interviews were carried out in two fire stations in San José (four sessions total), with a total of 20 firefighters interviewed. These interviews were undertaken over two working days per station, to allow information to be obtained from different firefighters due to shift work. The first interviews were carried out at the Barrio México station, where the interviews were carried out with individual firefighters. The second set of interviews were performed at the Pavas station, where the interviews were performed in groups (as per the station's preference). These two stations were selected due to the relatively large number of informal settlements they are responsible for in the case of fire, with Pavas being the station that handles the most informal settlement fire cases in the country.

The interviews were carried out in Spanish and audio-recorded, after which they were transcribed and translated to English. The questions were formulated to be semi-structured and open-ended. The questions prompted firefighters to share details from their past experiences responding to informal settlement fires, with particular attention on firefighting challenges, similarities/differences between fighting fires in the suburbs in contrast with informal settlements, and their observations and perceptions relating to human behaviour during these fires.

Table 1. Fire Fighter Survey Themes and Relevance.

Themes	*Relevance*
Causes of the fires	The main issues that cause fires in informal settlements
Fire detection and warning	Identifying systems that are used for fire detection and warning, and their effectiveness.
Response time	Identifying factors that affect firefighter response time to a fire incident in an informal settlement.
Fire containment and extinguishment	Identifying factors that affect firefighting efforts to contain and extinguish the fire, including comparison of attendance to fires in these regions versus fires in the suburbs.
Strategies and equipment used to fight fires	Identifying tools, techniques and products used for firefighting, considering possible differences in fire behaviour and access compared to fires in the suburbs.
Advice for communities	Determining how the Fire Corps engage with communities for fire prevention, including what type of advice or preventive measures they provide.
Experience in past events	Feedback of past experiences could help to better address the problem.

Table 1 shows a summary of the themes selected to formulate the questions and their relevance to the study. The following questions were then formulated and posed to the interviewed firefighters. These questions were chosen to inform the authors' understanding of pre-decisional and decisional behavioural responses of residents as well as the interacting actions of the firefighters:

- What are the main causes that start fires in informal settlements?
- What is the process and duration between fire detection and when the station is informed?
- What is the approximate response time it takes to service a fire in this area?
- Which factors influence the time it takes to put out a fire?
- What difficulties do you face in extinguishing a fire in an informal settlement as opposed to in the suburbs?
- How do you extinguish fires, what equipment, techniques or products are used?
- What preventive measures or recommendations are offered to the population inhabiting informal settlements?
- What would you propose to reduce fires in this type of settlement?
- Could you tell me about your experience with fires in this type of settlement?

2.2 RESULTS

The questions were formulated under specific themes as described in Table 1, therefore, the results are discussed herein under the same theme headings.

2.2.1 CAUSES OF THE FIRES

While some fires in San José informal settlements had unique causes, all of the firefighters interviewed indicated that many fires had causes related to the following:

- Electrical connections in disrepair and short circuits;
- Cooking with open flame (including irregular ground causing stoves to fall);
- Poorly maintained and manipulated gas cylinders;
- Arson;
- Dwellers performing welding tasks near combustible materials (e.g., mattresses);
- Attempted controlled burns of wires (for the collection of metal);
- Self-ignition of dry grass;
- Lit cigarette butts;
- Candles or wood-burning cookers left on without surveillance; and
- Children playing with matches, candles or left alone in their houses.

Firefighters also noted some informal settlement residents store hazardous items like buckets with sodium hypochlorite or pool chlorine. The interviewed firefighters have attended emergencies involving chemical reactions that can cause injuries (e.g., irritation to respiratory tract) or lead to fires.

Firefighters indicated there is a relationship between seasonality (time of year) and the occurrence of certain fire causes. They noted, for example, that most fires are related to non-controlled (self-ignition due to heat or sparked from other sources) grass burns which affect the surrounding structures in the dry season (December to April, low humidity). Similarly, in the rainy season (May to November), the most common cause of fire is short circuits in electrical connections presumably due to higher humidity. However, the frequency of fires in informal settlements and their causes are not recorded in a consistent systematic way, so these insights are based on firefighters' perceptions.

At the time of writing, only fires which meet the following criteria are recorded and investigated in Costa Rica: accidents that involve deaths; fires in hospital, penitentiary, public meeting, state buildings or educative centres; fires that burn more than $100m^2$; fires where the time required to control them surpass 30 minutes after the dispatch of units; and fires in structures with an insurance policy. Fire investigators try to identify fire origin, cause and other relevant information. This data is then processed with the Judicial Investigation Corps to verify it. The produced report is not typically shared with the public, unless a request is approved.

2.2.2 FIRE DETECTION AND WARNING

At the time of writing, the only emergency alert system in use in Costa Rica is phone dialling 9-1-1. This service is managed by Emergencias 9-1-1 Costa Rica and the dispatch of the fire units is managed by the Operational Communications Office (OCO) which belongs to the Costa Rican Fire Corps. When the 9-1-1 service receives a call, they either verify it by calling back the person who reported the incident, or consider it to be verified if they receive several calls from different people regarding the same incident. Once verified, the information is transmitted to the OCO, which communicates to the corresponding station via radio to dispatch the units. For informal settlement fires, the only difference in the emergency alert process is the verification of the call. When the call is received, the 9-1-1 service immediately communicates to the OCO and they make the dispatch. The verification process is carried out while the units are on their way. This procedure has been streamlined by firefighters because they acknowledge fire spreads in informal settlements faster than in other areas, such as in the 'formal' residential areas.

No information is available regarding informal settlement residents' awareness or willingness to use the 9-1-1 service. This is a research gap that could be explored in future studies including community engagement.

2.2.3 RESPONSE TIME

The firefighters interviewed estimated that five minutes elapses from the time a 9-1-1 call is received until units are dispatched via radio communication. The travel response time for units to reach the incident varies based on factors such as traffic, weather conditions, and the availability of the units. Access to the location of the fire can be a significant issue because the streets in informal settlements are often too narrow to allow fire truck access. Firefighters are therefore required to access the fire incident by foot, resulting with delays in the initial response and challenges with subsequent firefighting efforts.

The El Pochote settlement (described in Section 3.0), for example, can only permit the use of fire trucks along the exterior of the settlement. Alleyways that lead to the centre of the community can be as small as approximately 1m in width (based on video observation). Around 75% of the firefighters interviewed commented that the people living in informal settlements often guide them to the fire through paths which are unknown to the fire brigades, helping them to get to the site.

The firefighters interviewed did not indicate any issues regarding delays in response due to tension with the residents upon arrival. However, they did indicate it is common procedure to involve the police when responding to informal settlement fires, whose responsibilities include crowd control.

2.2.4 FIRE CONTAINMENT AND EXTINGUISHMENT

Firstly, it should be acknowledged that there is a complex relationship between the specific fire scenario, detection and alarm time, fire growth and spread, resident response and fire services' response. In other words, informal settlement fires, and all fires for that matter, are highly situational. This section provides an overview of technical aspects of fire response from the perspectives of firefighters. While it was not possible for the authors to engage directly with residents for this study, it is important to recognize their role in fire response, including before and after the firefighters arrive. Residents are the first responders in a fire event and their capacity (or lack of capacity) for communications, evacuation and firefighting will impact everything, from the development of the fire to their decision if/when to contact the fire services, and more.

It is also important to acknowledge how physical and socioeconomic vulnerabilities of informal settlement dwellers may influence behaviour before, during, and after a fire. For example, persons with disabilities, elderly persons, children, pregnant women, and others who may require assistance to escape may be more likely to evacuate away from the incident than to assist with firefighting.

Recovery from fire can be a long and difficult process, which can be exacerbated in informal settlements where financial insecurity, insecure tenure, and a lack of insurance are common. People may lose all their possessions, their home, and their livelihoods can be impacted. These factors can affect how people behave during fire incidents. In South Africa, for instance, it is common for residents to re-enter their dwellings to retrieve their most valuable belongings and to fight the fire without protective equipment or sufficient water resources [25]. The impact of retrieval of belongings on evacuation is a topic area which has been recognized as needing more research within the Human Behaviour in Fire research community [26][27][28].

The firefighters interviewed highlighted several factors that often make their response to informal settlement fires difficult and affect the time it takes to contain and extinguish fires.

Location. The dwellings in informal settlements are not always located near the streets, and if they are, the streets often do not have adequate dimensions for the fire units to access them. Therefore, attacking a fire entails creating long combinations of hoses. Sometimes the only entrance available to reach the fire are through narrow streets, which often have steep slopes and steps made of soil or wood. Firefighters use a range of technologies including aerial drones to plan access during incidents.

Structure characteristics. The materials and methods of construction, dimensions, and the resulting structural stability can influence fire behaviour. Most of the houses in San José informal settlements have been constructed with flammable materials, as shown in Figure 3, such as wood, cardboard, plastic, or materials that may enable heat transfer like corrugated steel sheets (uncoated).

Figure 3. Typical Dwelling Configuration at La Carpio (Authors' photo. May 7th, 2019)

Other fuel loads within houses and settlements. Residents' belongings both inside and outside of their homes can act as fuel for the fire and contribute to fire behaviour and fire spread. Considering the small compartment size of many informal settlement dwellings, a mattress and basic furniture can be more than enough fuel to cause severe fires, with very short time to flashover, perhaps even less than 1 minute. Furthermore, walls are often thermally thin with significant leakage. Fire may spread to adjacent dwellings in just a few minutes. Combustible garbage and vegetation surrounding dwellings can also help to increase the severity of the fire and act as a bridge for fire spread between dwellings. If several dwellings become involved in a fire, fire behaviour sometimes appears to be akin to wildfire behaviour, with a fire line of several dwellings spreading through a settlement [5].

Access to water supply resources. Informal settlements usually lack basic services and city infrastructure, including access to fire hydrants with reliable water supplies. It is common for firefighters to respond to informal settlement fire incidents in locations without nearby fire hydrants, or where available hydrants do not have reliable water supplies. When firefighters have insufficient water, they depend on cistern trucks, which usually take more time to arrive at the fire scene. Multiple cistern trucks may be required for larger fires and delays in additional trucks being dispatched and arriving on scene may hinder firefighting efforts.

Topography and weather conditions. Some informal settlements in Costa Rica are on river edges, mountain slopes, or in places with rugged topography. Thus, some of the properties are on steep slopes and experience high wind speeds, which along with other weather conditions, can intensify fire spread.

Floor plans and lack of structural integrity. There are several differences between fighting a fire in informal settlements and in the suburbs. According to the firefighters interviewed, emergencies in suburbs are easier to manage since those dwellings are normally constructed following the country's construction codes and standards, thus, having stable housing construction and safer communities. The informal settlements are more difficult regions to attend to due to complexities associated with access and the features of the houses built.

The houses located in suburbs are divided by masonry walls, meanwhile, the informal settlement houses are commonly divided by thin corrugated metal sheets, which intensifies the likelihood of fire spread and collapse. However, a benefit of the metal sheet walls is that residents can sometimes deconstruct them during a fire to escape, whereby masonry walls can more easily trap residents attempting to evacuate during a fire. In suburbs, it is easier for the firefighters to rescue people trapped in the property on fire since these houses have more standard floor plans and are therefore easier to navigate compared to informal settlements dwellings. Firefighters interviewed noted they can usually clearly identify the number of people living in dwellings in the suburbs and therefore quickly figure out if search and rescue support is needed (and for how many people). In contrast, overcrowding is common in informal settlements and the number of people living in a dwelling is often unknown to firefighters, so thorough searches are needed. Despite these significant search and rescue challenges, firefighters need to work quickly because the poor structural integrity of dwellings in informal settlements presents a serious collapse hazard to firefighters and residents.

Interactions with residents. In both formal and informal settlements, it is reported by the firefighters that people's behaviour is very group oriented and collaborative in response to fire. Firefighters interviewed said that upon their arrival to a fire in an informal settlement, residents will most likely be attempting to control the fire themselves. They noted that residents usually want to support their firefighting efforts as well. Firefighters generally appreciate their willingness to help, and therefore give them orders so they can support fire response in a coordinated way. For example, residents may form bucket brigades to transport water to the fire and firefighters may direct them. However, the firefighters also commented that even when the action was helpful, these people sometimes risk their own lives and hinder firefighting efforts. Also, residents often take their belongings out of their houses and place them in the alleys or the streets, which further limits the space available to move and respond to the fire.

The firefighters interviewed noted they have observed aggressive behaviour more often in responding to fires in informal settlements compared to fires in the suburbs. These firefighters said they have received insults and threats, been stolen from, forced out of their fire truck upon arrival on site, and in some cases, physically harassed. Of the firefighters interviewed, 25% of them perceived that children partook in stealing equipment when this happened. While the firefighters interviewed did not provide insights to the potential triggers of this behaviour, one firefighter did note that residents sometimes seem frustrated that firefighters do not do what residents ask them to do. The Fire Corps response plans to informal settlement fires now include the support of the police on site due to these concerns.

2.2.5 STRATEGIES AND EQUIPMENT USED TO FIGHT FIRES

The main resource used to extinguish fires is the application of water, but on some occasions, firefighters use a combination of water and pressurized air to respond to the fire. The pressurized air is mixed with various quantities of water to produce different mist densities. Firefighters interviewed shared that the predetermined vehicular response for informal settlements is two fire trucks (each with a thousand gallons of water) and a paramedic unit. In cases where the fire cannot be controlled, reinforcements are then requested which include the dispatch of cistern trucks with more water if necessary.

Upon arrival, firefighters first observe the fire behaviour to identify which techniques should be used to control the fire. In Costa Rican informal settlement fires, firefighters mostly try to create fire advance control lines, which means they destroy the houses near the fire to make space and eliminate the surrounding fuel (i.e. fire breaks). This is performed to control the growth and spread of the fire. There was no indication from the firefighters to whether community members in Costa Rican informal settlements support this strategy, or not. Note, this technique has been observed in other countries – e.g., it is referred to as 'flattening' in South Africa and as the creation of 'spontaneous fire breaks' in Syrian refugee camps in Lebanon [12].

2.2.6 SUPPORT TO COMMUNITIES

Once the fire is extinguished, firefighters have observed residents helping their neighbours who suffered losses by sharing their belongings and providing comfort.

Immediately after the Costa Rican Fire Corps extinguishes a fire in an informal settlement, they engage with the affected community to share fire safety good practices. The information they share is mainly related to improving electrical connections, however other recommendations are given regarding behaviours and physical changes to their living spaces, such as:

- avoiding storing chemical products or fuels;
- avoiding having dry grass surrounding the walls;
- not leaving candles or open flames stoves unattended;
- improving gas connections;
- when performing welding jobs, informing their neighbours and having a fire extinguisher nearby;
- avoid leaving children alone; and
- implement the usage of flashlights instead of candles.

The firefighters, however, indicated that these recommendations are not usually implemented. They believed this is predominantly because residents do not have the budget to improve their condition.

To promote these preventative measures, the Fire Corps create campaigns, children's camps, prevention speeches, etc, with the main objective being to encourage prevention and raise awareness within these communities. Electrical companies also provide information, training, and recommendations regarding safety for electrical connections.

Following a fire, several governmental institutions such as the National Commission of Emergencies (CNE), Aqueducts and Sewers (AyA), the Municipal Committee of Emergency (CME), the Ministry of Housing and Human Settlements (MIVAH) and the Mixed Institute for Social Aid (IMAS) join with other non-governmental organizations like Obras del Espíritu Santo (Works of the Holy Spirit) to activate their humanitarian response protocols for affected families, providing them shelter, food, potable water, clothes, personal hygiene supplies, and subsidies, which may include payment of rent in a formal leased apartment for three months.

It should be noted that this information was shared from the point of view of the firefighters and related organizations. As there was no interaction with informal settlement residents in this study, it is unsure if they are aware of these resources, or their ability to access them.

3.0 EL POCHOTE CASE STUDY

To meet the objectives and aims previously described, the authors examined a case study fire that occurred on September 16th, 2019 at 4:19 pm, in the El Pochote informal settlement, located in Barrio Cuba, Hospital district, San José Province. The reported cause of the fire was a failure in the electrical system (not identified specifically to the powerlines, connection system nor a dwelling itself). For this fire, seven fire stations dispatched firefighters and equipment in order to achieve fire containment and extinguishment. Six of the stations were from San José and the other station was from the Alajuela province. Once the fire was extinguished, the Fire Corps reported a burned area of 2,400 m2, with 40 houses and 219 individuals directly affected, among which there were 70 children [11]. Figure 4 show the totality of the informal settlement, the blue lines show the boundaries which encompass an extension of 0.16 km2 and the red hatch represents the burned area.

Figure 4. El Pochote informal settlement and the representation of the burned region (shown in red) [29].

Through the Costa Rican Fire Corps, the authors obtained 71 minutes of real time video footage of the fire, as well as five aerial drone videos (10 minutes each) taken by a journalist from the San José Municipality showing the damage post-fire (provided to the authors upon request for publication). The authors received written permission to use the videos for research purposes from the owners (the Fire Corps and the journalist) and university ethics clearances were then performed where needed and appropriate. The films were blurred to remove possible identification of individuals for publication.

The raw videos were analysed in order to better understand the behavioural actions during this fire by residents and emergency response crews, as well as their interactions. The authors attempted to objectively appraise the behaviours of firefighters and inhabitants observed in the videos, while avoiding subjective comments that can mislead the reader's interpretation. Section 4 will provide further discussion. The interviews presented above also enabled comparative analysis of firefighter perceptions with observed actions from the videos.

Surveys and interviews with the population inhabiting the informal settlements were not conducted mainly due to ethical and safety concerns relating to a lack of existing relationships and appropriate resources to support the engagement. Furthermore, the authors' institutions did not grant safety and ethics clearance to enter the settlement, as the fire was perceived to potentially be criminal before the fire investigation was completed [10] and it was not permissible to take an investigative approach post-fire in the site for researcher safety consideration. It is also possible that residents of the El Pochote settlement may not have wanted to contribute information, considering the then ongoing investigation into the fire's potentially criminal nature. If they did share and it was of a criminal nature, sharing their opinions and information could leave them exposed to retaliation and fear for their welfare. For all the reasons described above, interviews with residents were not carried out. The authors recommend engagement with residents should be included in future studies if safety and ethics can be ensured.

To provide context for the behavioural study described in subsequent sections and the discussion to follow, it is first necessary to describe the settlement's socio-economic background and the fire severity seen in the settlement. Despite the efforts of the authors to holistically undertake the study of this community and this fire, it must be recognized that there is only limited information available regarding the background, sociodemographic data, and physical features of this community.

3.1 SETTLEMENT CHARACTERIZATION

The socio-economic and political context of the El Pochote settlement is critical to understand the fire incident studied. Each settlement is unique, and researchers must seek to understand how specific contextual factors, such as regional cultural factors, influence decision making and behaviour (both daily and during a fire incident). The most recently available sociodemographic data of the population inhabiting El Pochote informal settlement was from the last National Census undertaken in 2011 [30]. A more contemporary survey is needed to draw demographic-specific conclusions, although the earlier census is still useful in gauging a general understanding of demographics within informal settlements in Costa Rica.

The census indicates that overall, there are approximately 2000 people living in El Pochote, with an average of around 3.8 dwellers per household. Of the whole population, approximately 21% have at least one disability documented (the type of disability is not described in the census). Furthermore, there are an average of 2.5 children per family. The population consists of 8% seniors (65 years or older). Among other socio-demographic indicators, 41% of the population are unemployed, and only 59% of the population have high school education. Half of the population has internet access via a computer, and approximately 75% have cell phones. All dwellings have water and electrical access (which cannot be said for all informal settlements in Costa-Rica), however only half are documented to have sewage access.

The census [30] also provides information to the condition of the dwelling structures: 20% of the homes are considered to be in a state of disrepair; 42% are considered to be in regular state; and 38% are considered to be in good condition. The condition statements are subjective though as the census does not provide information to what qualifies each condition state. Due to the inconsistent nature of the structures, the ventilation conditions, construction materials and building geometries of each unit are highly variable, resulting in varying fire loads and leading to significant difficulty in estimating the potential fire dynamics. Of the dwellings themselves, 35% are rented, 7% are borrowed, and the remainder are owned by the residents [30]. The census does not give indication if ownership of the dwelling implies land tenure as well.

There is no specific information regarding on the possessions present within each dwelling, which could provide a better understanding of a typical fire load in a settlement. However, it was observed in the video footage described later that an abundance of furniture and technology (TV and computers) exists within the settlement.

3.2 FIRE CHARACTERIZATION

The fire within the El Pochote settlement started due to a short circuit within the south-western corner of the burned area. The authors hypothesize that this fire could have been more severe without the previous day's rainy weather which pre-wetted the exterior of the buildings and reduced the likelihood of ember-induced fire spread. The wind also directed the fire towards a main transportation road (potentially serving as a fire break). On the day of the fire, the wind was blowing with a maximum speed of 12 km/h in the southwest direction, carrying the fire to the front road and the entrance of the settlement. Wind speed and direction can influence the spread of the fire, via transport of flaming (pre-heated) materials (embers from combustible materials) and by the fire plume orientation itself. If the wind had blown in the north direction, it likely would have spread the fire in that direction due to the number of houses with overlapping proximity to each other (exact measurements are not available).

To describe the size and severity of the fire from different angles, drone footage was referenced. These pictures and videos show the fire damage footprint, and some of the activities performed post-fire by the inhabitants of the settlement. One of these videos provides a view from above, with the drone beginning near the ground and increasing in height until the full fire footprint can be observed. Figure 5 shows the damaged footprint in detail.

Figure 5. Post-fire footprint with a burned area of 2,400 m2. (Drone footage photo by Jason Fernández, September 17th, 2019 with permission)

Examining these videos enabled the authors to analyse the materials left after the fire, including construction materials, and the general condition of the settlement. The videos show the following building materials which are typical to informal settlements in San Jose: masonry, rusted metal sheets, wood, steel tubes and steel hangers. Alleyways and pathways in the settlements are mud with protruding stones.

3.3 METHODOLOGY

The Costa Rican Fire Corps captured cell-phone camera footage (71:40 min) during the fire, recorded by a member of the Strategic Communication Office. It showed the arrival to the site of the fire units and continued until the situation was considered to be controlled by firefighters. The video shows both firefighters' actions, settlement dwellers' actions, and the interaction among them.

To contextualize the video that was analysed, Figure 6 illustrates the informal settlement in this study. It illustrates the road names that have been established by the authors for descriptive purposes. Figure 6 also locates an area described by the firefighters as 'Sector 3', which is composed of a roofed alley created by surrounding houses. The video is in Spanish and required translation by the authors post-filming.

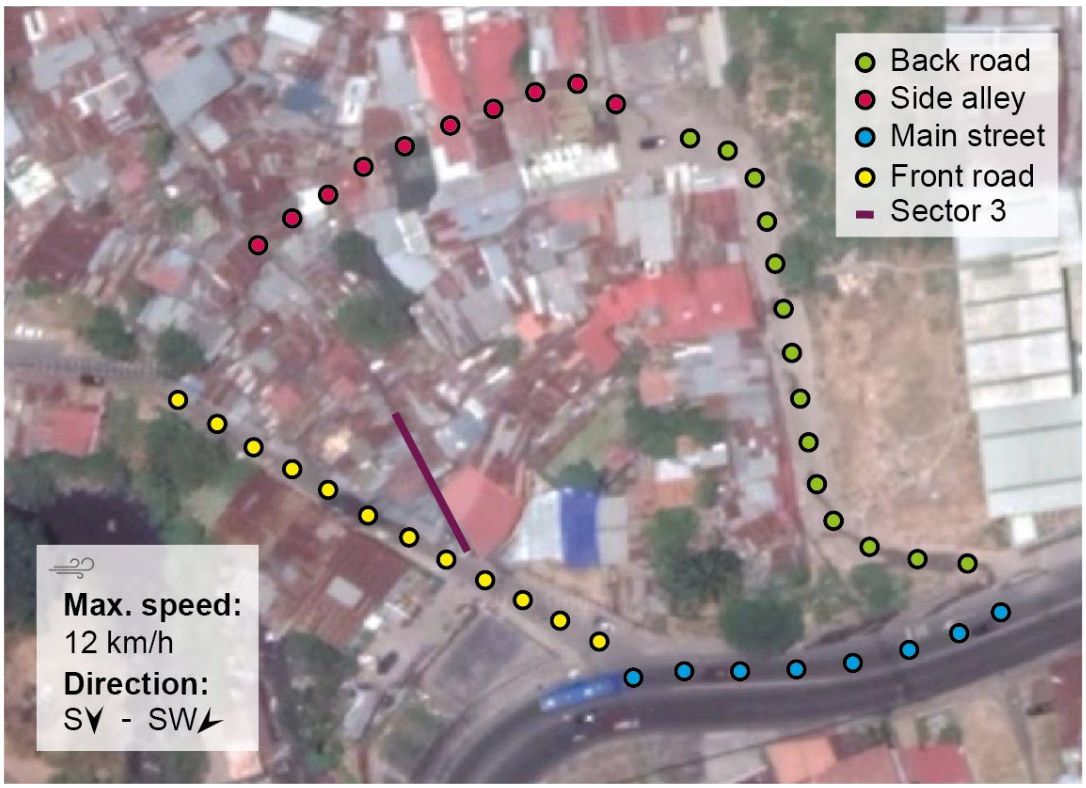

Figure 6. Wind speed and author given names of the roads of the El Pochote informal settlement fire. [31]

In the video, the cameraperson moves into several points of the settlement at different timestamps, allowing the authors to observe and describe the magnitude of the fire at determined moments and places. It should be highlighted that the start of the video occurs when firefighters are on route to the scene and does not coincide with the time of ignition of the fire. It was not possible to incorporate any insights to the fire's timeline before the arrival of the firefighters.

3.4 RESULTS – FIRE BEHAVIOUR

A brief description of the development of the fire will be made using images within this section followed by descriptions of behavioural and management observations in Section 3.5. Fire development is critical context that can influence behavioural actions of residents and firefighters. Screenshots are provided herein to help the reader understand fire development, but many of the observations described are not clear in only screenshots so description of the video footage is provided to allow much deeper insights.

Figures 7 - 13 shows the path followed by the cameraperson. The filming path was divided equally into spans of ten minutes, resulting in seven respective sets of images (I-VII). For example, Image I shows the first ten minutes whilst VII shows the last ten. Furthermore, the numbered dots on the associated maps show the locations were the camera stood at sequential key events.

3.4.1 Path I, illustrated in Figure 7, begins when fire units are just arriving to the settlement and starting to develop the strategy to contain and extinguish the fire. At point 1 (02:44 min), dense grey smoke and flames on the roofs can be observed. At point 2 (05:11 min), in the background of the video, flames appear at the height of the ceiling. At point 3 (06:48 min), flames engulf at least six dwellings. Later, at point 4 (08:34 min), there is a wider view of the event where the smoke plume is completely vertical, indicating in that moment the wind is not blowing strongly in a specific direction.

Figure 7. Screenshots of key events in Path I.

3.4.2 Path II shows the path along of the front road of the informal settlement. Figure 8 below illustrates the points described. At point 1 (10:50 min), electrical flashes can be seen through the smoke, and sounds resembling short-circuits are heard. At point 2 (12:33 min), flames can be seen over several roofs behind the smoke. Beside the smoke cloud on the left, a thin black plume begins to surge in the southeast direction. At point 3 (13:38 min), grey smoke in the sky can be observed. At point 4 (16:44 min), the vegetation on the houses catches on fire and the flames surpass the roof level. The first houses that were on fire (point 3, path I, Figure 7) are now collapsed. At point 5 (18:43 min), it can be observed that the fire is spreading quickly since approximately 8 houses are on fire (more houses than in Figure 7 path I point 3 which is the same location). Finally, at point 6 (19:58 min), the houses which recently caught fire had flames over the roof and the fire is spreading.

Figure 8. Screenshots of key events in Path II.

3.4.3 Path III shows the main street and the front road of the informal settlement. Figure 9 below illustrates the points described. At point 1 (21:20 min), the smoke plume was dense and dark grey, and it was moving towards the south. Some suppression activities illustrate spraying of water along the ceiling. The hose stream penetrates the (weak) ceiling and the cameraman gets wet. At point 2 (25:38 min), grey smoke can be seen, and the fire appears to be decreasing in intensity. At point 3 (26:12 min), around 60% of the houses observed to be burning are now extinguished. At point 4 (27:33 min), the cloud of smoke looks scattered but dark. At point 5 (28:19 min), a wider view of the event can be seen. The smoke in the sky is light grey and scattered. Later at point 6 (29:56 min), another plume of a dark grey colour is accompanied by flames appearing in the right corner of the screen. At that moment, the fire is at the entrance of the front road of the informal settlement.

Figure 9. Screenshots of key events in Path III.

3.4.4 Path IV At point 1 (30:57 min), very light grey smoke can be faintly seen. At point 2.a (31:30 min), flames and a dark, dense smoke column can be observed at the right side of the screen. Also, a less dense and grey cloud of smoke can be observed at the left side of the screen. At point 3 (35:55 min), most of the fire is extinguished and only 20% of the remaining area is on fire when compared to point 1, path III, Figure 9. At point 4 (36:36 min), the fire seems to be under control compared to earlier (point 6, path II, Figure 8), but the houses on the left side of the fire are now on fire (left side of point 4). The smoke appears disperse and light grey. The cameraperson returns to point 2.b (38:27 min), where the fire at this location is now controlled when compared with the first description of point 2.a at the same location. Figure 10 below illustrates the points described in this subsection.

Figure 10. Screenshots of key events in Path IV.

3.4.5 Path V covers the main street, the back road, and the side alley of the informal settlement. Figure 11 below illustrates the points described. At point 1 (40:38 min), grey smoke is observed. On closer inspection, at point 2 (41:20 min), flames can be seen at the smoke's source. The hidden alley seen in the background is the region given the name of 'Sector 3' by the firefighters. Then at point 3 (45:35 min), firefighters and residents are standing, and light grey smoke can be seen. At point 4 (47:25 min), the alternative alley to reach the fire from the back road can be seen. It has stone steps and walking is uneven and difficult. At point 5 (47:59 min), light grey smoke in the alley is at head height.

Figure 11. Screenshots of key events in Path V.

3.4.6 Path VI follows a path back to the start. At point 1 (53:57 min), grey and scattered smoke can be seen on the screen. Then at point 2 (55:53 min), the fire is almost extinguished after the last observed reignition of the fire (point 2, path V, Figure 11), but smoke is still observed. Later at point 3 (56:55 min), the fire is almost extinguished in comparison to how it was (point 4, path IV, Figure 10) with an estimated 10% of the fire remaining. At point 4.a (57:10 min), the fire is almost extinguished with just an estimated 25% left at this location, and several houses appeared to have collapsed. The house on the left is completely involved in fire. At the same location, point 4.b (58:11 min), the alley is fully covered with white smoke, which is moving up the street. Figure 12 below illustrates the points described above.

Figure 12. Screenshots of key events in Path VI.

3.4.7 Path VII The cameraperson is back to the location he started. Figure 13 below illustrates the points described. At point 1 (62:27 min), the fire is already extinguished but smoke is still observed. At point 2 (68:05 min), there is light grey smoke that looks sparse in the sky. Finally, at point 3 (70:13 min), the cameraperson declares the fire as being under control and then finishes the transmission.

Figure 13. Screenshots of key events in Path VII.

3.5 RESULTS – BEHAVIOURAL RESPONSE AND MANAGEMENT ACTIONS

The available video footage enables for a timeline to be developed and for the analysis of the behavioural and fire response actions performed by the inhabitants of the community, firefighters and even police officers. In this section, a brief description of this timeline with observed behaviours are provided. Analysis of the behavioural instances will be discussed in Section 4. It should be noted that the behaviour of the inhabitants could change in relation with the magnitude and severity of the fire, meaning that a different fire would show different human behaviour. Therefore, the data shared in this paper is only representative of this particular fire and of the human behaviour of a particular group of people at a specific moment of time. In a similar manner to the previous section, the description of the actions will be presented into spans of ten minutes to correspond to timings illustrated in Section 3.4.

Minute 00:00 to 10:00. At the beginning of the video, when the fire units are on their way, a man is giving directions to the fire units from the middle of the street. When the units arrive at the site, a group of approximately 45 people are gathered at the entrance of the settlement. They predominately are comprised of women, children and animals (dogs). Then, more firefighters arrive, including the chairman of the Fire Corps. As the fire is developing, men begin to help the firefighters, mainly extending the hoses (Figure 15.a). A man approaches the chairman and tells him "*You need to attack from the other side of the site, you just need to move 25 meters.*" The firefighter decides to go and check. Near the middle of the front road, there is a hallway, identified as 'Sector 3', which leads into the middle of the burning area. This hallway is covered with smoke and flames. A woman can be observed watching this hallway and talking on the telephone. After a while, she decides to enter (Point 2, Path I, Figure 7). She remains in the area despite the impending danger from the fire and smoke surrounding her. Later, near the end of the front road, around six local men are helping firefighters. One uses his shirt as face protection from the smoke, leaving his upper body unprotected. At the same time, several men are lowering a blanket with their possessions inside from the second story of a dwelling to the ground (Figure 15.b). A fire is in the adjacent dwelling. Possessions are stored in an annex just adjacent to the front entranceway. Several women stand near these possessions and are communicating on cell phones.

Minute 10:01 to 20:00. Many fire units and firefighters are in the region, and local men continue to assist them with hose allocation – including young boys who attempt to help after seeing older men try. It should be noted that the people helping are all male (predominately appearing between the ages of 20 and 35 years old). At this point, the police arrive on site and begin to direct civilians out of the fire affected region. Residents start to retreat beyond a plastic caution tape defining a perimeter that the police are setting up. This perimeter had the intention of clearing the region, allowing the firefighters to move and work freely. At one point in the video, a policeman says, "*Let's park the motorcycles near the firefighters' trucks, we need to prevent vandalism*." This indicates that they are there to keep the population in control and the fire units safe. A line of police motorcycles is seen later in the video footage. During this time interval, the police officers (20 of them) begin forming a perimeter line that is continually relocated away from the fire affected region. The perimeter slowly moves the residents who had already evacuated away from the settlement to a point where they would not be able to see into the settlement (see Figure 15.c). The police officers appear not to let people return, and when they see a resident near the firefighters, they begin to guide them to the perimeter line.

Minute 20:01 to 30:00. This time span (and the next) is mostly related to the management done by the Fire Corps and the police. Firefighters can be seen working on the fire with administrative firefighters (members of the Costa Rican Fire Corps who do not fight fires but oversee office paperwork) supporting them by bringing hoses or oxygen tanks. At one point, the chairman of the Fire Corps approaches the middle of the front road, in the hallway previously identified as 'Sector 3'. He begins to assess the sector and decides to work inside this hallway; firefighters can then be seen going back and forth to 'Sector 3'. Meanwhile, outside the settlement, police officers are closing the main street and setting up another perimeter further back.

Minute 30:01 to 40:00. At the beginning of this time span, a firefighter can be seen running towards the entrance to receive orders from another firefighter. This one is asking for reinforcements and says into the radio: "*Send me all the water you can*." At the entrance, some firefighters that had been working since the beginning of the emergency are taking off their equipment to rest.

Minute 40:01 to 50:00. In this lapse of the video, the cameraperson moves to the side alley. When he is walking towards the side alley, he says that "*Due the lack of hydrants, at the beginning of the fire, the water supply was given by several enterprises near the area, then the source of water was received by the cistern trucks*." When he arrives at this region, some men can be seen together taking care of their belongings, including furniture and clothing, on one side of the alley. Afterwards, policemen that were near the site begin trying to evacuate the dwellers but are ignored. Moving deeper into the alley, a group of men (approximately 30) can be observed moving back and forth to the fire affected region carrying buckets (Figure 15.d). Following this, a man yells "*we need more hoses*" and begins to run. Behind him, there are two men following him to help. Deeper in the alley, a man is climbing a roof and starts to run over the rooftops (Figure 15.e). The people at the border of the alley are looking around, and they tell the guy on the roof that he should watch out for the electrical wires near his head. Later, a firefighter moves into the area with more hoses, so the men in the surroundings begin to pull, extend and connect the hoses. Then, more men who were watching approach to help, but the firefighter shouts "*People! Order!*"

Minute 50:01 to 1:00:00. It is observed that getting out of the settlement is difficult. The configuration of the alleys looks like a labyrinth and the cameraperson comments to this effect and is lost. He receives help from the residents and some policemen until finally he returns to where he was before entering the side alley. When he goes out, another perimeter has been raised at this entrance by the police, and some officers are guarding the region and the belongings that the residents left there. One of the men that was previously helping with the bucket brigade is arguing with an officer who is preventing him from (re)entering. He says that the firefighters sent him to perform a task and now he is trying to deliver the message. It appears urgent and he thinks the message is essential, but the police do not let him through.

Minute 1:00:01 to 1:11:40. Finally, the fire is mostly controlled, but the firefighters are performing secondary investigations to identify remaining hot spots and to collect further information on the fire spread. One administrative firefighter is reviewing the area with a drone (Figure 14). At this point, some firefighters are getting out of the fire affected region, and the cameraperson says that everything seems under control and the transmission ends.

Figure 1. Aerial drone use.

Table 2 illustrates sample behavioural actions as observed to be taken by the residents which are seen throughout the video footage. Caution should be taken when consulting the table below, as the identification of types of behaviour can be subjected to bias and subjectivity. To remove subjectivity, multiple research team members reviewed these events separately, then the observations were compared. Similar observations were noted. It is acknowledged that this approach has limitations, as further discussed in Section 5. It should be noted that these are in no way a comprehensive listing of all behavioural actions occurring or all actions seen on film. For example, many cognitive biases associated with decisional stage actions would require surveying residents to gain insights to their decision-making process. As the film is only from one perspective, quantitative counts of each action are not listed. Instead, a listing of behaviours as observed to correspond to the various stages of the film seen is provided.

Table 2. Sample Resident Behaviours.

Sequence time in videos	Resident Behaviours Observed
I (0-10 minutes)	• People taking their possessions out of dwellings (these include tables, fridges, stoves, TVs, food bags, plastic containers) trying to save them from the fire (examples seen include the carrying of a full couch by one individual). • Family roles are apparent as it is observed that men attempt to fight the fire, children attempt to help men, and women tend to stay with the furniture which often included having a dog present. Women were also using cell-phones. (Figure 15.a) • Two men trying to lower down possessions from dwelling wrapped in a sheet. (Figure 15.b) • Re-entry into the fire affected region to collect items. This is stopped by assembled police force. (Figure 15.c)
I, II (0-20 minutes)	• Man telling the firefighter where he needs to work, firefighter follows. • People giving advice to the firefighters. • Man giving instructions to companions acting as if he knows how to extinguish fire. • Residents following the orders of the firefighters and police officers. This involves staying behind a perimeter that was established by the police.
II, III (10-30 minutes)	• Children see men helping firefighters with hoses, they try to help as well. • Inhabitants start to remove their belongings after they have seen their neighbours do so (identified through proximity by the authors). They do this even when there is no direct threat of fire to their dwelling.
IV (30 to 40 minutes)	• Group of men and young boys begin to help firefighters with water-buckets (forming a water bucket brigade) after first resident (a man) begins – residents appeared to copy his behaviour though it is unclear to the order of actions that may have occurred off camera. Buckets range in size – small buckets which are practical for firefighting up to drums which are too large to practically move and use for firefighting. (Figure 15.d) • Men and young boys helping the firefighters without any personal protective equipment, some of them cover their faces with their shirt which leaves their bodies uncovered.
V, VI, VII (40 to 70 minutes)	• People moving back and forth in hallways or houses without apparent stress.

	• Man climbing a roof, exposing himself to potential danger from fire or electrical wires. (Figure 15.e) • Children playing in playground with women. • People looking around, watching the fire behaviour and how it is developing.

The following image depicts the behaviours described in the paragraphs and the table presented above.

Figure 15. Behaviours observed from the video footage.

4.0 DISCUSSION

While some resident behaviours observed appear to be consistent with existing behavioural framework and behavioural theories, the authors have not classified these behavioural actions directly under contemporary behavioural frameworks due to a lack of appropriate evidence from this fire and from other informal settlement fires. Some behavioural frameworks like PADM use methods based on the processing of information from social and environmental cues in the early stages of emergency detection. Others, such as heuristics, the transactional stress model, and security motivation system are also mostly applicable to pre-decisional stages [32]. In the present case study, the video footage does not include imagery from the beginning of the fire, thus, there is little evidence that allows the authors to classify the actions performed by the inhabitants using frameworks reliant on pre-decisional behaviour. To validate a provisional PADM theory in the context of informal settlement fires, for example, more diverse case studies would need to be analysed, which rely on both video footage and engagement with residents, to better grasp the full pre-decisional and decisional processes over a range of fire scenarios. As such, using the data from the present case study, a particular theory or framework could not be validated. The data, however, is useful to build upon and should be considered in future research. As for other theories, such as the Theory of Planned Behaviour (TPB) that has been applied to building fire evacuation, the authors found that they do not apply to unplanned behaviours like the ones observed in the video footage of the case-study. Finally, the hazard to action chain model theorem is not considered applicable for the case-study (at least not yet), since it has not been determined if this model is suitable to fire emergencies [32].

Notwithstanding behavioural theories that cannot be linked to El Pochote case-study, there are several behaviours observed in the video footage that are similar to behaviours seen in other fire incidents. As Thompson et al. [33] note in their review of dwelling fires, where evidence of people attempting to tackle fires, movement through smoke, re-entry behaviour, and clear differences between males and females in behaviour and frequency of certain behaviours (males were more likely to exhibit firefighting behaviours whereas females were more likely to alert others and exit the property) has been observed.

More precisely, Table 2 in the previous section illustrates a number of unique behaviours as observed relative to the stage of the video. Internalization or the need to collect an individual's possessions or objects seems predominate in the early stages of the video (sequence I). It is not clear whether these actions depend on the pre-decisional mindset of the resident, are influenced by the fire itself, or depend on the knowledge that the police officers will eventually stop their access to the site. At this initial stage in the video footage, a gendered response is noted, which appears to follow roles somewhat independent of age of the resident (men are fighting the fire, where women are guarding possessions and children). This observation is then followed by a clear indication of a positive normative social influence regarding residents' firefighting efforts (II, III, IV). Actions such as removing belongings altogether, and the formation of bucket brigades are more specific positive examples of a normative social influence shared amongst the residents. The remaining footage demonstrates multiple instances of information processing and the milling of groups of people regarding the situation (V-VII). These are only a sample of the behavioural actions exhibited in what appears to be the decision-making stage of the fire event. The behaviours do exhibit a sequential operation of protecting possessions after the defence of their and their neighbour's structures. It would appear that the authorities' crowd control influenced the role the residents played.

It should be noted that the behavioural responses of the population observed in this paper are only specific to the fire event on September 16th and to the degree to which it was recorded on film. Most behavioural actions by residents were in response to the events occurring at that instant, as observed within portions of the footage. Other portions of the footage showed operational measures being taken by the firefighters. For example, there is limited footage of evacuations taking place from the settlement itself. Most of the footage shows re-positioning of belongings to controlled areas by the firefighters and localized resident evacuation from dwellings. The observed evacuation appeared to be carried out by approximately 20 police officers on site, who managed a controlled perimeter, which was re-positioned approximately every 5 minutes to move residents approximately 10m further away from the area affected by the fire. Most of the evacuation process began prior to the cameraman arriving but continued during filming.

When it comes to evacuation dynamics, it was observed that upon arrival of the cameraman, approximately 45 people were at the entrance way. These were predominately women and children, as well as people with visible disabilities (wheelchair users for example) and various animals (dogs). The entrance way to the settlement was also used to store possessions. A more severe fire may have provoked different resident responses as well as management of the fire, however, speculation about such differences is beyond the scope of this paper, as all the informal settlement fires are unique. Consequently, this can affect the response of the population and the way the firefighters manage the situation. Furthermore, it is not possible to generalize the observations made in this case study to other informal settlements without more case studies to compare to.

The authors also observed similarities of behaviours to those reported in other literature sources on informal settlement fires. For example, Cicione et al. (2019) performed interviews with firefighters in the Western Cape, South Africa, and hypothesized that informal settlement dwellers are often afraid to share how a fire started if it was related to arson or to negligence on their part, because they fear retaliation from other residents, e.g., someone might set their dwelling on fire [34]. Also, Kahanji et al. (2019) stated that if an individual knows that they were linked to the start of a fire, they may be unwilling to admit such details, even if accidental, due to fear of reprisal from neighbours who have lost their homes [25]. Similar behaviours can be attributed to the residents of El Pochote regarding information contributions about the fire ignition causes, according to interviewed firefighters.

Other similarities with the case study were found in the research about the 2017 Imizamo Yethu (IY) South Africa fire, described by Kahanji et al. (2019) [25]. In this paper, the observations identified the tendency of the inhabitants of the IY settlement to store amounts of hazardous items, such as bottles of kerosene in their homes, and the involvement of the local residents in both trying to save their possessions and trying to evacuate the area, making the access and escape highly restricted. In both fires (IY and the El Pochote case study), the firefighters reported a number of verbal and physical abuses by the local residents during the incident. In the IY fire however, a resident cut a hose being used by firefighters and redirected the water onto their own dwelling [25]. This was not observed in the inhabitants of El Pochote during the fire. It is known that during these conflagrations, an effort to have a police escort was made in both fires. Comparing the answers received by the firefighters interviewed in this study and the commentaries stated in Kahanji et al.'s research paper, it was noted that both fire brigades have experienced theft when attending fires in informal settlements.

Other behaviours, however, were observed both by the firefighters as well as in the video footage. This allowed for the validation of the information gathered through the interviews since most of the behaviours observed aligned with the perceptions of the fire brigades. For example, as stated by the firefighters, residents took out their belongings outside their houses and located them over the alleys and they were also moving around the fire zone impeding the free movement of the firefighters, leading to the lifting of the perimeter by the authorities. Nonetheless, from the constructive perspective, the community rapidly formed a bucket brigade with the main intention of helping the firefighters to tackle the fire. During the video, the firefighters began to work with them as a team, since both shared the same goal. Finally, it was observed that several inhabitants were trying to get information risking their life, as the woman going back and forth into an alley on fire, or the men walking over the roofs or trying to help to extinguish the fire without any protective equipment that was available to the firefighters.

The methodology used within this paper is provided for other researchers to follow when collecting information regarding human behaviour in fire events in Costa Rican informal settlements and elsewhere. For the documentation of future case studies, it is proposed that following the steps used herein will assist others for comparing with the authors datasets:

1. Stakeholder engagement should be made with both firefighters and the local community. While in the case of this study, it was not possible to engage the residents for confirmation of certain behavioural actions, there was supplemental data generated by the fire brigade, and valuable footage of the fire incident was studied. In the event of video footage to be analysed, subjectivity must be reduced through screening by multiple researchers. Video footage including as much of the settlement as possible and from multiple vantage points may capture more behaviours.
2. Contextual data on demographics and socioeconomic status of the community, from a census for example, is critical to inform analysis of human behaviour.
3. A description of the conditions and informal settlement features should be gathered, such as access, configuration, street widths, alleys, and building make and condition.
4. The fire dynamics observed in the fire event should be described and if possible be explained, including the possible influence of environmental conditions.
5. The timeline of the event and the response by the fire brigades and other intervenors should be depicted, as well as analysis of the human behaviour of the residents.
6. Post-event surveying with community stakeholders to explore behavioural actions observed.

5.0 LIMITATIONS AND FUTURE WORK

The limitations faced during the research process were mainly related to the absence of information available to the authors. There is an extensive gap of knowledge regarding human behaviour in fires in informal settlements, which includes:

- Lack of research and understanding of the subject. There is very limited literature related to human behaviour during fires in informal settlements, and insufficient evidence and insights to observed behaviours and key factors influencing human behaviour during fire emergencies;
- Lack of appropriate behavioural theories and frameworks applicable to this context. The relevance of existing theories cannot yet be evaluated due to limited evidence available on informal settlement fires, and limited research attempts to carry out this type of work; and
- Absence of statistical fire incidence data, and data on sociodemographic and economic factors of informal settlement communities. In Costa Rica, these databases are not

updated by any institution, and (to the authors knowledge) no one gathers the background or physical features of this settlement.

It is therefore not possible to apply definitive contemporary behavioural frameworks and theories to this case study at this time. Future research should consider testing a range of behavioural models from those illustrated earlier (and others), identifying where and in what sense they may be applicable. If they are not applicable, future research may propose a behavioural model unique to informal settlement fires that can begin to incorporate the range of behaviours seen in multiple communities. It is also considered by the authors that it would be a good practice to analyse some frameworks previously developed, as frameworks related to the application of forensic fire investigation principles to informal settlements [35] among others can be useful for the creation of a framework suitable for the specific region. By developing such a framework, it would then be possible to create a fire response plan that stakeholders could utilise to mitigate and prevent life loss, injury, and excessive damage. Insights from behavioural response could inform investments in fire risk reduction – mitigation, preparedness, and recovery, as well as response.

Another limitation was regarding the viewpoint of the community members since this study did not include the survey of the inhabitants of El Pochote informal settlement. This should be considered in future studies but may be prohibited due to the complex socio-governance of the communities themselves as well as stringent ethical considerations.

A limited overview of fire behaviour has been supplied to provide context of the behavioural response. There is a lack of research and understanding of enclosure fire dynamics in informal dwellings, which are highly heterogeneous, and with fire behaviour of informal settlement fires in general. In this study, it was observed that informal dwellings did not maintain integrity from the firefighters' hose streams during the suppression phase. Some of the dwellings filmed exhibited post flashover characteristics with abundant ventilation from gaps in construction driving the fires (see Figure 16). These dwellings often do not have formal planning (from the structural and fire safety perspective), excessive ventilation and use poor materials and construction techniques, which make them difficult to study.

Figure 16. Dwelling fire in distance, showing ventilation gaps and post-flashover behaviour.

Another limitation was regarding the subjectivity attached to the identification of the behaviours. The authors analysed this video individually, permitting each member to determine the actions that they observed, then the actions were discussed and matching behaviours between analyses were selected. While the authors were being objective, there was no strict methodology related to information gathering and reliability analysis from video footage, as used by Kopenhaver-Haidet et al. which used computation reliability coefficients to measure the internal consistency when rated by different individuals [36]. The information gathered herein provides a descriptive qualitative account of the events within this context.

It is important to reiterate that the observations seen herein may not be generally applicable to all informal settlements in Costa Rica or internationally. Every informal settlement is unique, as is every fire. Various socio-economic differences regionally make comparisons particularly difficult in Costa Rica, as well as varying fire brigade response practices which are highly dependent on community relations. It is encouraged that future researchers should carry out more investigations into fires in informal settlements, including different fires in the same settlements and fires in different informal settlements. This would help to improve the entire research community's understanding of fires in informal settlements and enable research on potential risk reduction interventions, which could be used for the development of local Community Fire Safety Strategies.

Further research is needed to investigate methods of firefighting used by communities and to learn about informal training that takes place between community members with regards to fire safety, even if it is not referred to as fire safety training – e.g., a mother teaching child to not play with matches or to go near the stove when she is cooking.

This paper is a first stage. It aims to support future research, being a foundation upon which additional evidence of human and fire behaviour in informal settlements can be compiled. Future researchers are encouraged to collect more information about fire response and behavioural aspects that can be detected when larger fires occur. The authors acknowledge that the fire in this case study is not as severe as it could have been under different climate conditions (drier conditions, unfavourable wind direction, etc.). It is understood that the fire was relatively smaller in size compared with other fires that have been documented in Costa Rica in past years in other informal settlements. For example, in November 26th, 2016, the informal settlement “Los trillizos” located in Leon XII district, Tibás canton, experienced a 3000 m2 damage region leaving more than 180 people without housing and had six fatalities. On January 15th, 2020, the “Los Sauces” informal settlement located in the Guararí neighbourhood, which belongs to the San Francisco district, inside Heredia canton, had a fire that destroyed 189 houses covering an area of 8400 m2. The El Pochote case study herein was only 40 houses located in 2400 m2.

Once additional data is compiled, the authors and others will be able to address a more specific fire response framework for the community. Proposing one now for this specific region would be premature. Future work may draw upon other disasters for a resiliency approach should it be applicable, and it should also consider those proposed elsewhere and how they are applicable here. It would, however, be a premature endeavour with only one fire case study to draw upon to enable a framework with supplemented data from other disaster types or other regions where settlements have been more extensively studied.

6.0 CONCLUSIONS

This research paper analysed human behaviour in a fire in an informal settlement. Analysis was based on a video footage of a fire in El Pochote, located in San José, the capital of Costa Rica. The response principles used by the firefighters to control the fire, the behavioural actions by the local inhabitants during the fire, and the interactions between the firefighters and the locals were explored in this paper. A number of aspects that make it difficult to study fires in informal settlements were discussed in this study, including the complex social interactions which are highly situational, such as density of the informal settlements, their expansion, education, unemployment and other demographics that may play a role in human behaviour in fire. As well, it is generally known that many fires in informal settlements are never reported to the fire department because they are managed by the community, and further firefighting support is not needed.

This study contributes to informal settlement fire research by providing a foundation upon which additional evidence of human and fire behaviour in informal settlements can be gathered and compared. While behaviours of the residents were often consistent with the existing behavioural frameworks, e.g. PADM, the authors observed that the current frameworks lack empirical evidence to be applied to human behaviour in informal settlements. Nonetheless, these theories can provide a base upon which researchers can build and form theories applicable to the context of informal settlements. To this extent, this paper outlined and undertook the preliminary research steps needed to show how future studies can better examine human behaviour in informal settlements.

Several key findings were presented that pertain to the behaviours of the firefighters and the locals. According to the members of the Costa Rican Fire Corps, civilians intervene in their fire extinction efforts, thus the firefighters have opted to work alongside the communities rather than fighting against their involvement throughout the fire containment and extinguishment process.

However, behaviours that are counter-effective, reported by the firefighters as well as in other studies, were not observed in the El Pochote fire video footage.

Nonetheless, some resident behaviours were observed to mirror those in the existing literature, such as attempting to tackle fires, movement through smoke, re-entry behaviour, and clear differences between males and females in behaviour and frequency of certain behaviours (males were more likely to exhibit firefighting behaviours whereas females were more likely to alert others and exit the property). In addition, novel bold actions by some residents, such as walking on the roofs, passing alleys while firefighting operation was taking place, and blocking the alleys with personal belongings, were observed.

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