Guidance for the Post-Fire Structural Assessment of Prestressing Steel

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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].

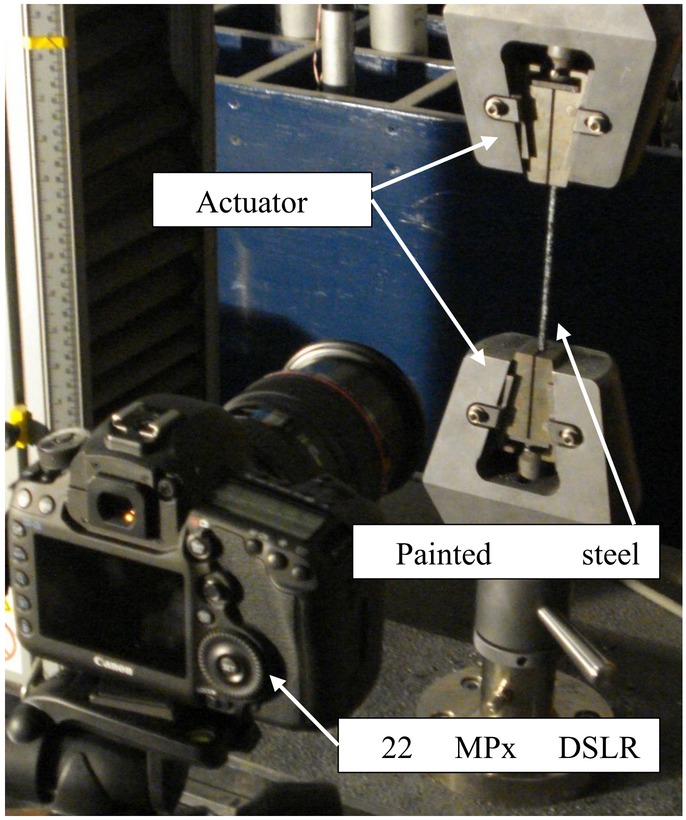

Figure 1. Loading apparatus with digital image correlation.

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.

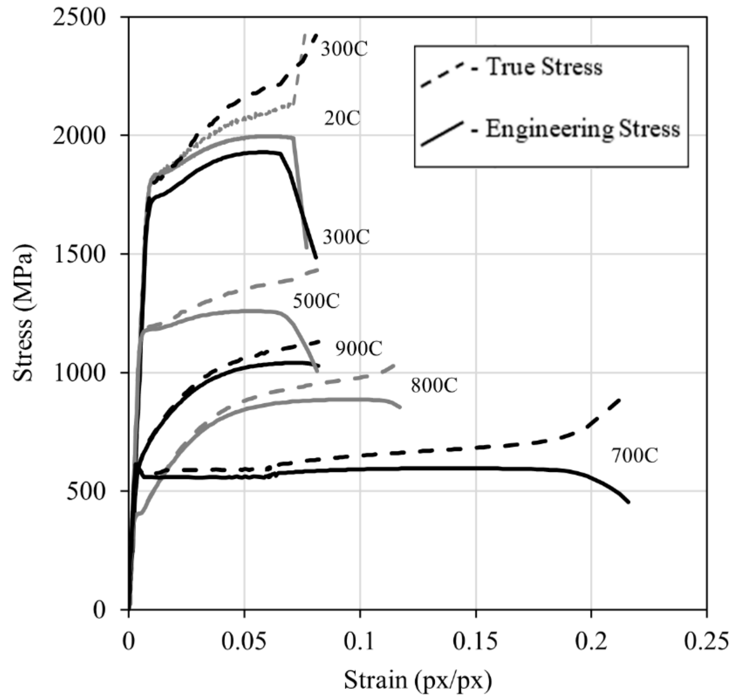

Figure 2. Stress-strain relationship when BS 5896 is heated for two hours.

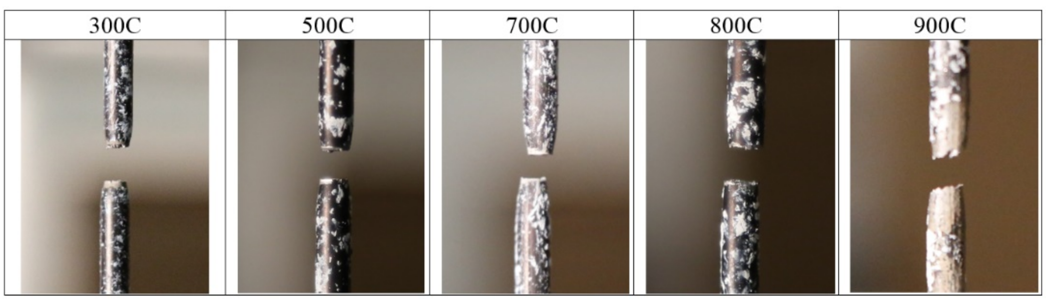

Figure 3. Fracture profiles for BS 5896 after two hours of heating.

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.

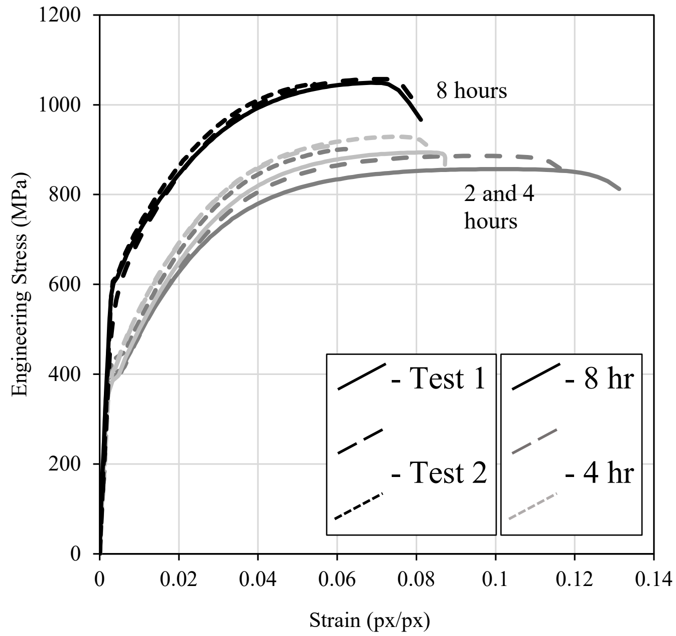

Figure 4. Engineering stress-strain relationship at 800°C for BS 5896 with repeat testing shown.

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.

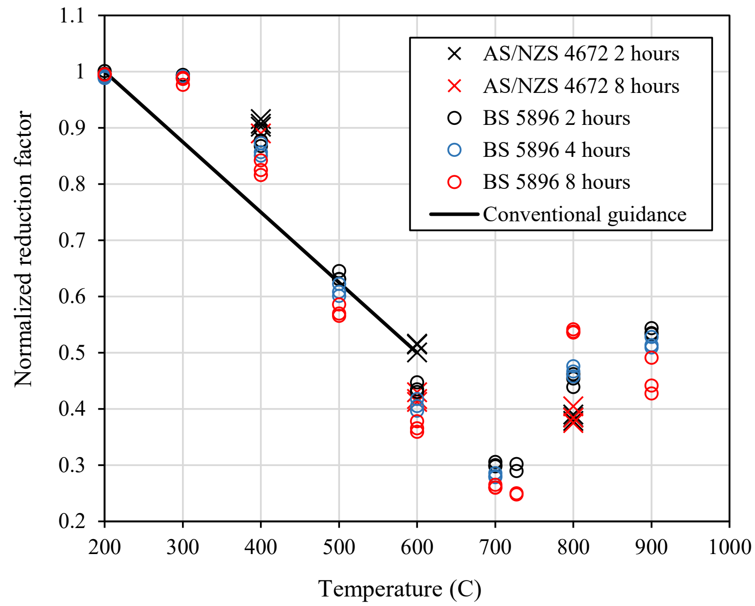

Figure 5. Ultimate strength reduction factors for stocks of BS 5896 and AS/AZS 4672 with conventional guidance after 200°C.

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.

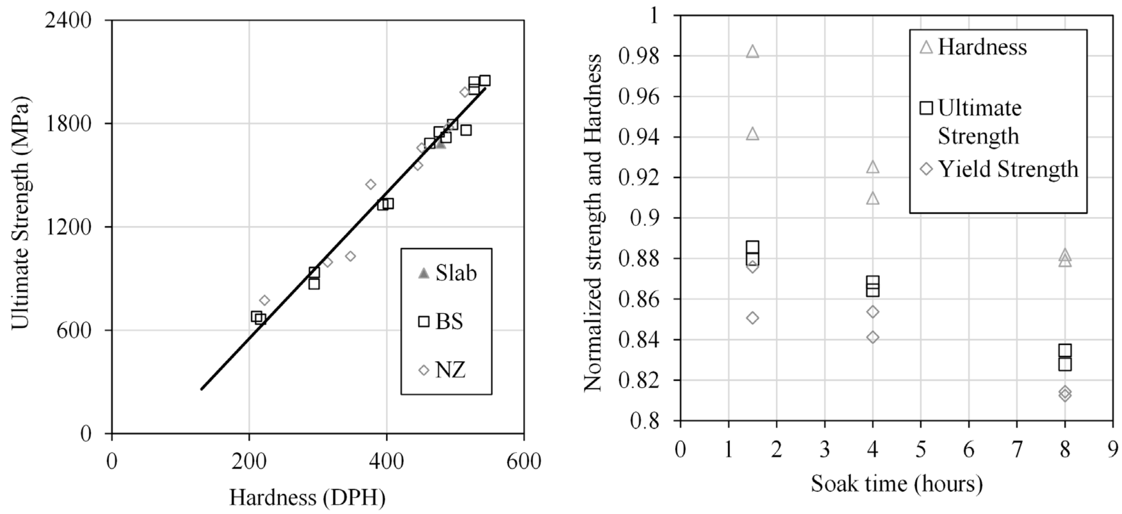

Figure 6. Hardness variation with (left) ultimate strength and (right) soak time at 400°C.

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.

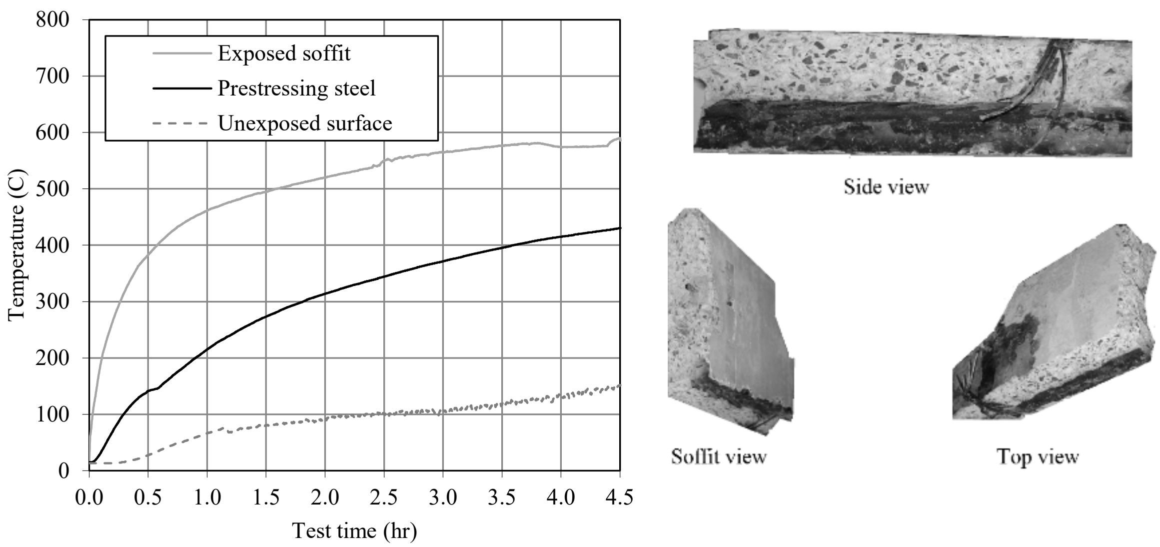

Figure 7. (Left) Measured temperatures for slab test and (right) an excavated slab.

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.

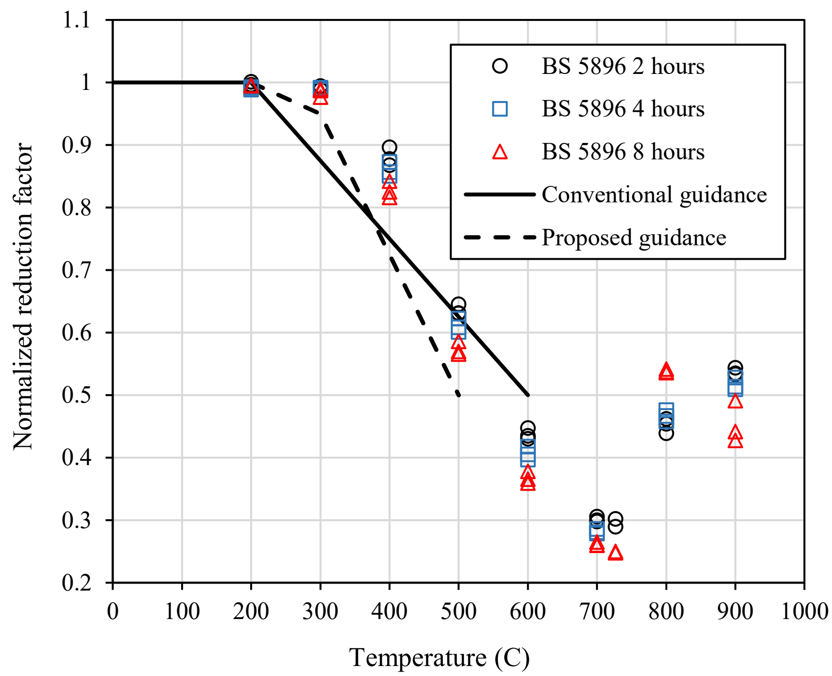

Figure 8. Contemporary prestressing steel strength reduction guidance.

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.

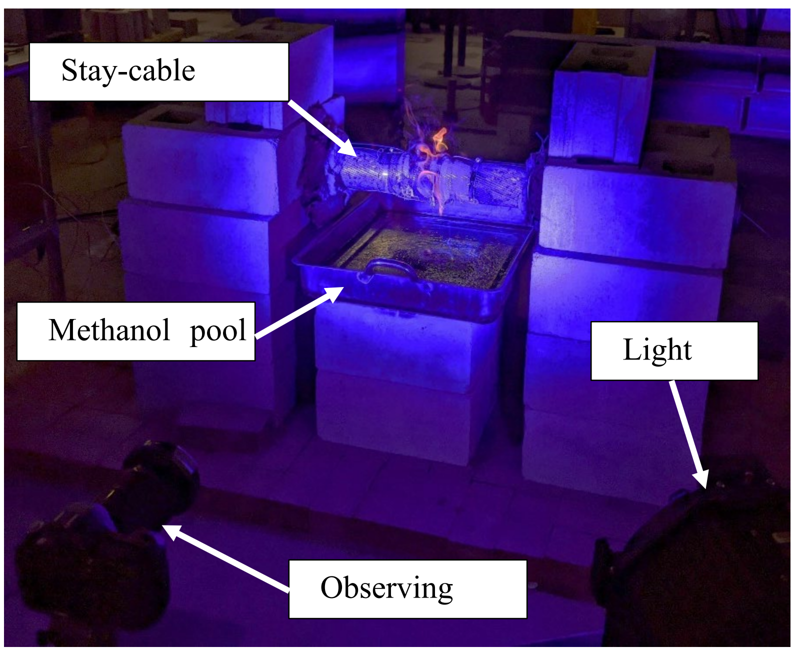

Figure 9. Experimental configuration of the tests conducted by [38]. Shown is a 100 mm locked-coil stay-cable.

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.

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