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Estimating In-Situ Flexural Strength of Heat-Affected Prestressed Concrete Beams Using Constituent Material Models

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ESTIMATING IN-SITU FLEXURAL STRENGTH OF HEAT-AFFECTED
PRESTRESSED CONCRETE BEAMS USING CONSTITUENT
MATERIAL MODELS

A Thesis
Presented to
the Graduate School of
Clemson University

In Partial Fulfillment
of the Requirements for the Degree
Master of Science
Civil Engineering

by
Travis Clayton Collins
August 2008

Accepted by:
Dr. Patrick J. Fortney, Committee Chair
Dr. Scott D. Schiff
Dr. Prasad R. Rangaraju

ABSTRACT

When a prestressed concrete structure is subjected to a severe fire, significant damage is created to the extent that a visual analysis is sufficient in rendering the structure unusable. Although moderate fires also create structural damage, the effects can often not be visually assessed. Therefore, a need exists for a method that allows structures to be evaluated after a fire has occurred and visual observations present no conclusive evidence for making decisions regarding the structural impact of the fire on the structure.

It was assumed that temperature histories of a member arising from a moderate fire can be determined using other analytical forensic techniques such as thermogravimetric analysis (TGA).

Experimental cross-sectional temperature profiles of prestressed concrete beams were measured for different temperature durations using a heating regimen that would represent a moderate fire. Based on these profiles, analytical constituent material models were generated using a cross-sectional fiber analysis program. The residual material properties of steel and concrete used were based on previous research. Moment-curvature relationships were generated for each heating duration. Finally, load tests were performed to measure force-deflection relationships of full scale heat affected concrete beams after cooling.

The results from this project indicate that a cross-sectional analysis program can predict the characteristics of a heat affected concrete beam within a reasonable degree of accuracy. The general degradation of strength and stiffness found from the experimental results were noted in the theoretical output. Based on this research, the structural integrity of heat affected prestressed concrete beams can be better estimated.

DEDICATION

I dedicate this thesis to my parents, Mark and JoAnne Collins, and my grandmother, Marie Collins, for their unending guidance and support.

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CHAPTER ONE

INTRODUCTION

Despite having the positive characteristic of being a poor thermal conductor, concrete remains susceptible in fires. High temperatures have a negative impact on its material properties, leading most notably to strength and stiffness degradation. After a fire, forensic engineers must determine if a structure, or individual members within a structure, has maintained sufficient structural integrity. In regard to reinforced concrete members, this can be a difficult task since post-fire visual inspections can be misleading.

Large, destructive fires that maintain high temperatures for long durations are not the scenarios that trouble forensic engineers. These fires create a substantial amount of spalling, large deflections, notable color changes, significant cracking, and sometimes failures. It is clear that use of the structure must be discontinued. The fires that present challenges for forensic engineers result from moderate fires. In the context of this report, moderate fires are those where relatively lower maximum temperatures impact the structural members, and create less notable visual effects. Minor spalling may occur, increased deflections that are still within allowable limits may be present, and there would likely be some color changes and thermal cracking. Since a visual analysis cannot gauge the extent of structural damage in such a scenario, forensic engineers are likely to turn to *The Guide for Determining the Fire Endurance of Concrete Elements* (ACI 216R-89-reapproved in 2001) to provide a quantitative means of evaluating the damage.

Similar guides are also available, such as the PCI Design Handbook (PCI Design Handbook, 6th ED), but essentially follow the same procedure.

ACI 216R-89 provides a method for designing and evaluating the capacity of structures at specific time durations *during* a fire. The majority of the design figures and material properties available in this manual are only applicable at high temperatures, with the exception of residual concrete strengths. Consequently, current guidelines are unsuitable for estimating the flexural strength of prestressed concrete members after having survived a fire. The main reason for this is that material properties of both steel and concrete are known to change upon cooling. Depending on the temperature attained, steel will often regain a large portion of its original strength. In contrast, most changes in concrete properties are irreversible and in some cases may even continue to decline as time progresses beyond fire extinguishing. These characteristics do not follow the standardized strength reduction relationships prescribed in the ACI 216R-89 and may lead to unnecessarily conservative estimates of *in situ* strength.

Therefore, a need exists for a method that allows structures to be evaluated after a fire has occurred and visual observations present no conclusive evidence for making decisions regarding the structural impact of the fire on the structure.

Current fire endurance standards for concrete structural systems are determined by the standard fire test as outlined in the ASTM E 119 standard (*Standard Test Methods for Fire Tests of Building Construction and Materials*). Specifically this standard, using the

temperature curve detailed in the ASTM E 119, gauges temperature loading and evaluations to determine the ability of a structural assembly to contain a fire. However, because this time-temperature curve does not represent real fire conditions, standard fire test results using these guidelines can only compare the endurance properties of concrete structures undergoing the standard fire event. They cannot provide insight into their structural performance during real fire events. Indeed, the ASTM E 119 guideline makes just this point:

“This standard should be used to measure and describe the response of materials, products, or assemblies to heat and flame under controlled conditions and should not be used to describe or appraise the fire-hazard or fire-risk of materials, products, or assemblies under actual fire conditions...”

From the perspective of both structural integrity, and performance, the ASTM E 119 standard fire test provides no practical data in regard to the *structural* performance of reinforced concrete, precast concrete, or prestressed concrete structures. If the critical condition of structural endpoint is reached during a standard E119 fire test prior to a flame passage or heat transmission end point, it is unclear if the tested member is able to retain its structural integrity since the fire load that caused the structural end point does not represent a real fire. In fact, the definition of the structural end point used for the fire test may not necessarily show any compromise in the integrity of the structure. In cases where structural concrete members “pass” the standard fire test, this endpoint information is only used for comparing similar types of assemblies. It provides no insight on the actual fire capabilities of the tested structure, nor of its structural integrity.

For reinforced, precast, and prestressed concrete, fire containment capabilities during actual fires are likely to be significantly higher than those measured during standard fire tests; maximum material temperatures in real fires for any given period of time may be significantly less than the standard temperature for the same given period of time (Copier 1983). Because flame passage and heat transmission end points in standard tests may not show the deficiencies of structural components to withstand accepted real-world fire scenarios, current ASTM fire testing standards impose overly conservative criteria. Thus, the ASTM E 119 standard cannot accurately assess fire risk in existing reinforced concrete structures. This is especially true when evaluating how well these structures comply with both code level performance and strength requirements. For this reason, a modified time-temperature curve was developed with which to represent realistic heat loads during moderate fire events.

1.1: Project Objectives

The overall goal of the research presented in this report is development of a method for determining the *in situ* flexural strength of heat affected concrete members using constituent material models. In this study, cross-sectional flexural strength will be evaluated using a cross-sectional fiber model where cross-sectional fiber properties will be based on changes in stress-strain relationships relative to the estimated propagation of heat into the cross-section. It is assumed that temperature histories of a member arising from a moderate fire can be determined using other analytical forensic techniques such as

thermogravimetric analysis (TGA). Assuming the maximum temperature and duration of the heat load can be accurately estimated, a cross-sectional temperature profile can be estimated using knowledge gained from this study, in addition to previous research, from which the fiber cross-section can be generated. With the cross-sectional model, appropriate concrete and steel properties are assigned to fibers throughout the cross-section, and strength as well as moment-curvature relationships are determined from which cross-sectional analysis is performed. The application of the methods developed are intended to serve forensic engineers challenged with evaluating the structural integrity of concrete structures where visual observations of the structure do not provide conclusive evidence as to the structural damage caused by the fire.

In order to achieve this goal, three objectives were established:

1. Develop experimental cross-sectional temperature profiles for different heating durations based on a modified time-temperature history relative to the standard time-temperature curve defined by the ASTM E 119 standard,
2. Create analytical constituent material models representative of the experimentally-measured cross-sectional temperature files, and
3. Load test full scale beams to measure force-deflection relationships of heat affected beams after cooling, and use to calibrate against analytical cross-sectional fiber models.

In order to generate temperature profiles, the maximum temperature and duration of the fire must be known. Past research has indicated that this data can be gathered through the use of thermogravimetric analysis (TGA). Although TGA techniques will not be evaluated as part of this project, the objectives will be based the assumption that TGA provides a reasonable method for accurately estimating the temperature history of reinforced concrete member after having survived a fire.

The modified time-temperature curve was created and used to more accurately depict a real fire where the surviving structure provides no indication of structural damage through visual observations. Cross-sectional temperature profiles were then measured using embedded thermocouples. Based on these profiles, an analytical constituent material model was developed using XTRACT (XTRACT, v3.0.5), a cross-sectional fiber analysis program. Each temperature layer within the cross-section's profile was assigned its appropriate concrete or steel properties based on previous research. Once this was achieved, cross-sectional flexural capacities were predicted, moment-curvature relationships were generated, and changes in elastic and inelastic EI slopes, as a function of heat from fire were evaluated.

Full-scale experimental beams were also heated for different time durations and then load tested after cooling. The experimental results were then compared to the analytical models. Calibration of the stress-strain relationships used in the analytical model were then made when necessary.

1.2: Project Significance

Results from this project will lay the groundwork for the development of guidelines for evaluating the residual structural integrity of heat-affected concrete members after having survived a moderate fire. Temperature profiles of structural shapes other than those evaluated in this report, may be generated, forensic engineers would then be able to use the forensic evaluation methods presented in this report, and combined with appropriate modified material properties, accurately diagnose structural degradation of structures surviving a moderate fire event.

CHAPTER TWO

LITERATURE REVIEW

2.1: Visual Analysis

Although there are more advanced and accurate means of measuring the maximum temperature a concrete member reaches, it can be desirable to compare these results with a more efficient method. Visual assessment is one way of doing so. One characteristic to investigate is the color change in concrete as a function of maximum temperature exposure. Previous studies have shown that concrete turns pink or red between 250 and 300°C and grey or purple between 500 and 600°C (St John et al. 1998). By comparing a heat affected concrete member to these characteristics, a maximum temperature can be estimated. Also, these color changes are not just seen on the surface of the concrete member. Instead they remain consistent throughout the cross section of the member, enabling an individual to estimate the duration of the fire based on the depth a color has penetrated.

However, these color indicators have proven to be unreliable for a variety of reasons; one of which is due to their dependence on aggregate type. For example, the pink color change is thought to occur due to the oxidization of iron compounds (St John et al. 1998). Thus, if a concrete mix does not contain aggregates with notable iron deposits, it may not exemplify these generalized traits. Carbonation can also create discoloration that may be mistaken for fire damage (Nassif et al. 1995). Finally, the depth a color penetrates the

concrete can be altered by spalling. Since spalling can occur while the fire is growing or when it is dying down, color depths cannot be accurately analyzed(Nassif et al. 1995).

Another feature to assess is the amount of cracking observed. Without magnification, cracking is first evident in concrete when it is exposed to approximately 1112°F (600°C). It then becomes more pronounced at 1472°F (800°C) and extensive at 1832°F (1000°C). After this point, the binding properties are lost and spalling occurs at 2192°F (1200°C) (Arioz 2007). Based on the color and cracking characteristics, the ultimate temperature a concrete member has reached cannot be estimated until approximately 572°F (300°C), which is after the onset of strength degradation.

More detailed changes are noted when viewing the sample on a microscopic level. Cracking is found around aggregate boundaries and within the cement paste when the concrete is above 932°F (500°C) as opposed to only localized boundary cracking below 572°F (300°C). The range between 572 and 932°F (300 and 500°C) is essentially a transition zone where there are boundary cracks and some intra-paste cracking. Since this intra-paste cracking occurs around 300°C, it is thought that this could be a significant cause of the onset of strength degradation (Riley 1991).

Similar to the analyses based on color changes, cracking characteristics also change based on the type of concrete mix. This is because different aggregates and cement pastes have different measures of thermal expansion. It is this thermal expansion that

creates internal stresses which in turn cause cracking (Riley 1991). Therefore, different expansion values will yield different magnitudes of cracking.

Spalling can occur before reaching the high temperatures mentioned above. Hertz (2003) states that there are many reasons for spalling to occur; one cause is rapid heating. Rapid heating can create large moisture gradients that can lead to explosive spalling within the first 20 minutes of a fire. However, if there is enough free water, rapid heating does not need to occur. While water tends to evaporate through the surface when heated, it can also penetrate further into the member where it is cooler. This will create a moister zone and can lead to a steam explosion. Another reason is due to the sloughing-off or corner spalling. Sloughing-off is a combination of cracking and thermal stresses that often takes place at the corners of beams and columns. The crack pattern will loosen a chunk of concrete enough that its weight will cause it to fall off (Hertz 2003).

2.2: Compressive Strength of Concrete

Although there are several factors affecting the correlation between temperature and compressive strength of concrete, it can generally be described as an indirect relationship. The many changes concrete undergoes when heated, such as the dehydration of cement paste and thermal expansion of aggregate, are thought to affect this strength (Abrams 1971).

Another variable affecting the compressive strength of heated concrete is its loading state (Abrams 1971). In order to simulate different scenarios a structure might encounter, three different tests are used. First is the unstressed test. This involves a specimen being heated to a certain temperature and then loaded immediately to failure. Next is the stress test. Here, a specimen is heated to a certain temperature while enduring a constant compressive load of $0.4f'_c$. After the desired temperature is reached, the specimen is then loaded to failure. The final test is the unstressed residual test, where a specimen is heated without being loaded and then allowed to cool to room temperature and stored for 7 days before being loaded.

It was found that the stressed specimens had a larger compressive strength at all tested temperatures (Abrams 1971). The reason for this may be that the constant pressure limits the amount of cracks formed from high temperatures (Malhotra 1956). Also, the amount of constant stress applied to the specimen did not have a notable affect on the ultimate capacity (Abrams 1971).

In contrast, the unstressed residual strength was lower at all tested temperatures. The strength degraded linearly to approximately 30 percent of the original strength at 1112°F (600°C) (Abrams 1971). Research performed by Caple (2007) showed similar results. In the Caple study, cylinders were heated to various temperatures for different time durations. Although the durations had a negligible impact, it was found that cylinders

held at 1112°F (600°C) for various durations exhibited approximately 30 percent of their original strength (Caple 2007).

Still another variable is aggregate type. Of the three types tested by Abrams (1971), carbonate, siliceous, and sanded lightweight, strength loss was found to be similar up to 900°F (482°C) in all testing conditions. However, beyond this point, the siliceous aggregate began to have a greater loss in strength. The reason is thought to be due to the deterioration of the bond between the cement paste and aggregate, occurring around 1000°F (538°C). The carbonate and sanded lightweight concrete remained similar in strength loss.

2.3: Static Modulus of Elasticity of Concrete

Similar to the compressive strength of concrete, the static modulus of elasticity decreases as the temperature increases (Cruz 1966). The compressive strength of the concrete and the aggregate type used did not have a definitive impact on the percentage reduction of the moduli of elasticity. It was noted, however, that the siliceous aggregate had a more linear reduction. In addition, the lower strength mix had a notable increase in the rate of loss when the temperature of concrete reached 900°F (482°C).

2.4: Tensile Strength of Prestressing Steel

Abrams (1961) investigated the behavior of prestressing steel at high temperatures. One relationship that was evaluated was the effect of the rate of heating on the rupture temperature of the prestressing steel. Two different prestressing forces were tested with three different heating rates each. It was found that there was no correlation between the two variables. The rupture temperature did not vary more than 10.6°C (19°F) in either set of prestressing forces. However, the prestressing forces did have a significant impact on the rupture temperature. The strands that were stressed to 55% of the nominal ultimate strength had rupture temperatures at least 90.6°C (163°F) greater than the strands stressed to 70% ultimate strength. Tests were also performed at prestress levels 40 and 85% and the results further illustrated that there is an inverse relationship between prestressing level and rupture temperature.

The effect of the rate of cooling on the residual tensile strength was then investigated (Abrams 1961). Two methods were used to cool the tendons: “slow cooling”, where the temperature was slowly lowered by furnace controls, and “fast cooling”, where the steel was quenched with cool water. In both cases, the tendons were unstressed and heated to temperatures ranging between 850°F (455°C) and 1400°F (760°C) and this temperature was held constant for 30 minutes. The results showed that the tensile strength of the tendons was not affected by the rate of cooling.

Abrams (1961) also investigated the effect of sustained heating on the rupture temperature of prestressed strands. Here, strands were stressed to 55% of the nominal ultimate strength and were exposed to room temperature, 400°F (205°C), and 600°F (316°C) for either 30 or 90 minutes. After the sustained heating duration was complete, the temperature was increased until strand failure. The results indicated that the rupture temperature is independent of the initial heating conditions.

Another important relationship examined by Abrams (1961) was the tensile strength as a function of temperature. Similar to the compressive strength of concrete, the tensile strength decreased as temperature increased. The strength degradation curve can be described as having an “S” shape, where the strength decreases gradually up to approximately 400°F (205°C), then drops dramatically up to approximately 1000°F (538°C), and then starts leveling out again. It was also noted that strand diameter had little, if no, effect on strength degradation.

The residual strength of prestressing steel degrades with increasing temperature. However, the temperature at which degradation begins and the rate of strength loss varies amongst past research. Research by Holmes et al. (1982) indicates that prestressing steel will regain nearly all of its tensile strength at maximum temperatures up to 392°F (200°C); once maximum temperatures pass this point, the strength will decline linearly down to 50 percent at 1112°F (600°C). This relationship was determined by heating 7-

wire prestressing strands to a maximum temperature for 30 minutes, allowing them to cool, then tested at room temperature (Holmes et al. 1982).

Neves et al. (1996) tested single prestressing tendons that were 5-mm in diameter. These tendons were held at the desired maximum temperature for 1 hour and were subsequently cooled by air or water jet. The results indicate that there is minor degradation up to temperatures between 572 and 752°F (300 and 400°C), after which declining to approximately 60 percent of the unheated tensile strength at 1112°F (600°C). In addition, there was little change in strength when the two cooling regimens were compared in this temperature range.

Results from recent research by Caple (2007) have similarities with Holmes and Neves. Here, the tensile strength remained fairly constant up to temperatures around 752°F (400°C), then began to decline to approximately 50 percent of the relative ultimate strength at 1112°F (600°C). Although the range of heating durations were significantly longer than Holmes and Neves, the different durations had a negligible impact.

2.5: Thermogravimetric Analysis

When concrete is heated, the cement paste undergoes dehydration reactions that are mostly irreversible and can be recognized by weight loss. This weight loss occurs gradually as a function of increasing temperature. Thermogravimetric analysis (TGA) is a method that utilizes known relationships between relative weight loss and temperature

to estimate the maximum temperature attained by another concrete sample (Harmathy 1968).

Results from research by Alarcon-Ruiz et al. (2005) show what causes these weight losses and at what temperatures they occur. The first weight loss occurred between 212 and 392°F (100°C and 200°C) and was due to dehydration of several hydrates. The second occurred between 842 and 932°F (450 and 500°C) and corresponded to the dehydroxylation of portlandite. Finally, the third main weight loss occurred at 1382°F (750°C) and corresponded to the decarbonation of calcium carbonate (Alarcon-Ruiz et al. 2005).

Caple (2007) tested the accuracy of TGA using concrete cylinders. Thermocouples were placed at the center of the cylinders and temperature gradients were estimated based on the interior temperature of the concrete and the exterior temperature of the oven. Samples were collected through the depth of the cylinder and it was found that TGA provides an accurate estimation of the maximum temperatures reached (Caple 2007).

It was noted by Harmathy (1968), Alarcon-Ruiz et al. (2005), and Caple (2007) that it was important to collect and test the samples within the first two days to avoid rehydration reactions from having a significant effect of the results.

CHAPTER THREE

EXPERIMENTAL TESTING METHODS

3.1: General Test Protocol

The beams used for the experimental portion of this study were 18 feet long 12 x 20 inches rectangular prestressed beams. A total of twelve beams were manufactured. One beam was reserved for measuring the cross-sectional temperature profiles, and the remaining 11 were used for load testing.

The beams were made using one of Tindall's standard concrete mixes that had a specified compressive strength of 6,400 psi. The beams were doubly reinforced with two $\frac{1}{2}$ -inch diameter bottom strands and two $\frac{3}{8}$ -inch diameter top strands. With the exception of the beam measuring temperature profiles, all beams had four closed #3 stirrups placed at each end, spaced at 6 inches on center.

In order to heat the beams, a stainless steel burner that was 2 feet wide and 5 feet in diameter was designed and fabricated. KAOboardTM lined the flat surfaces while KAOwoolTM was placed along curved surface with the purpose of insulating the system. With two propane blowers mounted tangential to the edges of the frame, heat circulated around the beam to create an even three-sided burn while avoiding having flames come into direct contact with the beam which could impose unwanted localized heating phenomena.

As mentioned above, an even three-sided burn was desired for the heating tests. Within the scope of this project, a three-sided burn can be defined as an even application of heat on the bottom and two side faces of the beam. It is assumed, however, that a slab exists and blocks the heat flow from reaching the top portion of the beam. A realistic scenario where a three-sided burn would occur is shown in Figure 3.1. Similarly, one, two, and four-sided burns exist and can be defined in a similar manner. For example, a four-sided burn would exist where heat is applied evenly on all four sides, and meant to represent a situation where a fire is able to engulf the entire beam.

Figure 3.1: Three-sided burn

In order to measure cross-sectional temperature profiles, six sets of thermocouples were spaced at intervals along the length of the beam such that a 2-foot wide burner could heat each set individually. Although the pick-ups (a steel hanger used for handling of the beam) would not allow for even spacing, the grids were arranged such that a gap of at least 8 inches existed between each burn area. Two sets were subjected to 3-hour burns

and the remaining four sets were subjected to 4-hour burns. Also, in order to test the effects of different cooling methods on the internal temperatures, half of these tests were air-cooled and half were rapidly quenched with water. Burner temperatures and cross-sectional temperatures were recorded every second in order to capture accurate temperature histories.

Two types of thermocouples were used and were chosen based on temperatures to which they would be exposed. The control thermocouple, used to measure the burner temperature, was a 24-gage, type-K thermocouple with Nextel Ceramic insulation. The thermocouples used to measure the temperature profiles within the beam were 24-gage, type-J thermocouples with glass braided insulation. The type-K thermocouple was rated for 1200°C while the type-J was rated for 800°C.

The data acquisition software used was Labview (Labview v8.2). A program, as shown in Figure 3.2a, was written that would allow streaming data to be viewed graphically, in order to see the change in temperature over time, as well as digitally, so that the exact temperature was known. All data was exported into text files.

(a)

(b)

Figure 3.2: (a) front screen of Labview program, and (b) DAQ Chassis

The hardware used to collect the data, as shown in Figure 3.2b, were NI-9211 4-channel, 24-bit, $\pm 80\text{mV}$ thermocouple input modules plugged into a NI cDAQ-9172 chassis.

Beams used for load testing were assigned labels based on the order in which the beams were cooked and the length of the heating phase. For example, the label B1-2 would indicate that this was the first beam in a group of 2-hour burns. The beam used to measure cross-sectional profiles was designated as B0 and each burn along the length of the beam was assigned a lower case roman numeral. For example, B0iv-4 (beam B0) would indicate the fourth burn location (iv) along the length of the beam, and a 4 hour (-4) heating duration.

Once the beam was prepared for the burn and the heating duration was determined, the burner could be placed onto the beam. Since the beam was elevated higher than the top edge of the opening in the burner, it had to be lifted with a fork lift. Wooden 2x6 inch boards were used to extend the legs of the fork lift so that it could lift both sides of the

stand evenly. After the burner was in position, the elevation was maintained by either leaving the fork lift in place or by sliding blocks under the casters. The method used was dependent on the availability of the fork lift.

The extension wire was hooked up to the thermocouples and to the thermocouple input modules. In order to verify proper sensor connectivity, a test was made before each burn where a flame would be held to the burner thermocouple, and the sensor response evaluated. Additional KAOwoolTM was added to the gaps between the sides and top of the beam surface and the burner opening. An air gap was left at the bottom of the burner's side openings to provide proper ventilation of the burner. Photos of the burner setup are provided in Figure 3.3.

Figure 3.3: Photo of burner installed on beam

Finally, the fans and propane were turned on and the burner was ignited. The propane pressure was controlled by an in-line regulator valve installed at the exit valve of the propane tank. Minor adjustments in the pressure regulator valve were found to be sufficient in accurately controlling burner temperature.

The remaining 11 beams were divided into four groups based on the heating durations to they were subjected prior to load testing. These groups included unheated (control) beams, 1-hour burns, 2-hour burns, and 4-hour burns. Due to the odd number, only two beams were tested as controls, while the rest of the groups were comprised of three beams each.

The load tests were carried out by placing monotonic cyclic concentrated loads at the midspan of each simply supported beam. The beams sat on two concrete columns with rocker plate assemblies, whose pin-to-pin spacing was 16 feet 6 inches. The loads were applied using a single-ended actuator which hung from a portable portal frame. A standard load history was used. Loading was stopped when the beam was determined to have gone beyond its practical ultimate flexural strength.

3.2: Concrete Preparation

A total of 12 beams and 50 cylinders were manufactured at Tindall Corporation. Due to space restrictions, the pour was divided into 2 days with half of the beams and cylinders being made each day. A standard mix design, as shown in Table 3.1, was used.

Table 3.1: Mix design for concrete beams and cylinders

Material	Quantity per cubic yard
Type IIICement	605 lb
Class C Fly ash	100 lb
Coarse Aggregate (#57 Stone)	1733 lb
Natural Sand	1155 lb
Water	234 lb
Air	5%

The beams were manufactured as doubly-reinforced beams, as can be seen in Figure 3.4, consisting of the following: two $\frac{1}{2}$ -inch diameter, 270 ksi, 7-wire bottom strands pulled to 70 percent ultimate strength, two $\frac{3}{8}$ -inch diameter, 270 ksi, 7-wire top strands pulled to 11 percent ultimate strength, and four closed stirrups at each end of the beam spaced at 6 inches on center. The two $\frac{3}{8}$ inch diameter top strands were present primarily for stirrup placement, and therefore, were pulled only enough to ensure proper stirrup placement.

Figure 3.4: Typical beam design layout

3.3: Beam Used to Measure Temperature Profiles (Beam B0)

One of the 12 beams had a slightly different reinforcement layout; no stirrups were provided, and the pick-ups were relocated to avoid conflict with the locations where profiles were to be measured. This beam, designated, B0, was used for cross-sectional temperature profile measurements. Temperature profiles were measured at six different locations along the length of the beam and numbered using lowercase roman numerals, as shown in Figure 3.5. At each of the six locations, a set of thermocouples were placed in this beam such that there was at least 8 inches of concrete between each burn area (see Figure 3.5 for the thermocouple layout used at each location). This was done to limit the effects of heat from one test on adjacent testing areas. The irregular spacing of the thermocouple sets was created so that the pick-ups found at either end of the beam would not enter the burn area. The pick-ups were excluded from the burn regions due to concern that they would potentially transfer heat at a faster rate than the concrete.

Figure 3.5: Thermocouple beam design layout

Each set of thermocouples was held in place by a wire mesh. A 19-gage galvanized steel wire was used so that it would be strong enough to hold the wires in place while having

minimal interference with the concrete settlement. Loops approximately $\frac{1}{4}$ inch in diameter were created at each thermocouple location to eliminate slipping during concrete placement. The thermocouple wires were also securely taped to the mesh. There were a total of three columns and four rows distributed evenly throughout the cross-section, as shown in Figure 3.6. Rows and columns were labeled so that thermocouples could be identified with a grid nomenclature.

Figure 3.6: Thermocouple layout

The wire meshes were secured by connecting each corner of the mesh to the prestressed strand using wire ties. These ties were tightened around the tape used to secure the thermocouples so that the thermocouples would not be damaged. This can be seen in a photo shown in Figure 3.7.

Figure 3.7: Wire mesh attached to prestressing strands

Since the internal thermocouples were not designed to withstand the temperatures reached within the burner, they were extended through the beam past the burn area. Therefore, after the meshes were secured, a 2x4 was clamped to the concrete form, 15 inches away, with a hole drilled in the center. This was used in order to have a secure place to feed the thermocouples out of the beam.

It was also necessary to control how the concrete was placed into the forms. In order to avoid placing any large forces on the wire meshes from one side or another, the concrete was poured as slowly as possible between each thermocouple set to a depth of approximately 1 foot. To further reduce the possibility of thermocouple damage during

pouring, 12x12 inch sections of plywood were cut to place over the meshes to prevent any concrete from falling directly onto them.

During beam manufacture, the 6-inch diameter by 12-inch tall cylinders were cast according to ASTM C 31 (Standard Practice for Making and Curing Concrete Test Specimens in the Field). The cylinders were then placed on a wooden palette adjacent to the beams to ensure they cured in the same environment. After the beams were removed from the manufacturer's forming beds, they were stored in the holding yard at Tindall along with the cylinders for approximately five weeks before being transported to Clemson.

3.4: Heating Preparation

All beams were cooked using a custom propane burner, as shown in Figures 3.8 and 3.9, designed and manufactured by Clemson faculty and technical staff. The burner consisted of the following components: a stainless steel cylindrical shell that was 5 feet in diameter and 2 feet wide, KAOboardTM insulation lining the flat surfaces and KAOwoolTM lining the curved surface, two propane blowers with single-speed fans, three hinged doors that allow it to slide on and off a test beam, and a stand with casters allowing for burner portability. The blowers were mounted tangential to edge of the burner so that the flames would directly contact the beam surface; instead it would be "cooked".

Soup cans with $\frac{1}{4}$ inch diameter holes were also attached to the ends of the blower muzzles which acted as carburetors for propene/air mixture. Additionally, three steel angles were placed in the bottom of the burner to hold concrete cylinders. See Figure 3.10 and 3.11.

Figure 3.8: Side view of burner

Figure 3.9: Front view of burner with doors open

Figure 3.10: Interior of burner

Figure 3.11: Concrete cylinders placed on steel angles in bottom of burner

Since a three-sided burn was desired, a 4-inch slab needed to be simulated along the top of the beam. In order to achieve this, KAOboardTM was glued to each burn area using high temperature cement and sealant. The board was cut such that it stretched the entire width of the burner, as shown in Figure 3.12.

Figure 3.12: KAOboard™ used to simulate presence of a 4-inch thick slab

In addition to the KAOboard™, the thermocouple wiring also needed to be prepared. Thermocouple extension wire was cut in lengths of approximately 20 feet for each embedded thermocouple in order to maintain a safe distance from the burner when collecting data. Cold-junction connectors were used to make the extension wire-to-thermocouple connection.

After running all of the tests on the beam with the embedded thermocouples, it became evident that the surface temperatures under the KAOboard™ would be required in order to generate the cross-sectional temperature profiles. Therefore, two additional J-type thermocouples were placed under the top and side KAOboards when heating the beams that would be loaded. This had to be done before cementing the boards to the beam.

3.5: Modified Time-Temperature Curve

Following the ASTM E 119 standard time-temperature curve would have likely created such significant damage to the prestressed beams that a quantitative analysis would not have been necessary to conclude that they were structurally inadequate. Since this was not the goal of the project, a modified heating regimen was created that would have a lower maximum temperature. The objective of this was to model the effects of a moderate fire where there was no clear visual evidence of structural damage. The modified heating curve followed ASTM E 119 up to approximately 347°F (175°C); after this point, the slope of the modified curve began to decrease, eventually leveling off after 90 minutes at 1112°F (600°C) as shown in Figure 3.13.

Figure 3.13: Time-temperature load curves; comparison of ASTM E 119 standard curve to target time-temperature load curve

Three phases can be identified within each temperature load curve. These were the ramping, heating, and cooling phases, as shown in Figure 3.14. The ramping phase was the portion of the curve where the temperature was increased until it reached 1112°F (600°C) after 90 minutes. After this point the heating phase began, where the maximum temperature was held for a specific amount of time. The length of this phase is how durations are identified. For example, a 4-hour burn defines a temperature duration in which the beam was held at 1112°F (600°C) for four hours. The constant heat load phase is 3 hours and 30 minutes long, 30 minutes less than what its name indicates. After the beam had been cooked for the desired amount of time, the cooling phase began. At completion of the heating phase, the burner doors were opened and the burner was then

moved away from the beam. Once the burner was clear, the beam and cylinders, if applicable, was either air cooled or quenched with water.

Figure 3.14: Three phases of heating cycle (Beam B0iv-4 shown)

The initial heating plan for B0 is shown in Table 3.2. This initial plan varied due to the uncertainty of how the beam would react during heating. The first two profile measurement tests were 3 hour burns and were located at the two center sets of thermocouples; locations i and ii (see Figure 3.5). The reason the two mid-span locations were chosen was to avoid the potential for strand slip at the ends of the beam where the bond stresses are greatest. Although spalling did occur at these two locations, there were no signs of strand slip or beam deflection. Next, the thermocouple sets located approximately 5 feet from the ends of the beam were tested with 4-hour burns (locations iii and iv; see Figure 3.5). Spalling occurred at these locations also, but there were still no signs of strand slip or beam deflection. The lowest heating duration was applied at the

two end sets of thermocouples (locations v and vi; see Figure 3.5) since the bond stresses would be greatest and chances of slipping are drastically increased. During the burns at locations v and vi, there were signs of minimal strand slip; however, there was no spalling. Since there was still no spalling by the end of the heating phase and no drastic changes in deflections or the amount of slipping, the heating duration was changed to 4 hours in order to gain more temperature data. The final heating plan is shown in Table 3.3.

Table 3.2: Initial heating plan for beam with thermocouples

Burn Identification	Heating Duration (hrs)	Cooling Method
i	3	Air cooled
ii	3	Quenched
iii	4	Air cooled
iv	4	Quenched
v	2	Air cooled
vi	2	Quenched

Table 3.3: Final heating plan for beam with thermocouples

Burn Identification	Heating Duration (hrs)	Cooling Method
i	3	Air cooled
ii	3	Quenched
iii	4	Air cooled
iv	4	Quenched
v	4	Air cooled
vi	4	Quenched

The heating plan, as shown in Table 3.4 for the remaining nine beams that were load tested followed a similar pattern. One beam from each set of heating durations was initially intended to be quenched. However, the KAOboardTM was knocked off while

installing the burner on Beam B3-1. In lieu of reattaching the insulation, the beam was tested without insulation as a four-sided burn and air cooled in order to compare it to the other two 1-hour burns.

Table 3.4: Final heating plan for load tested beams

Burn Identification	Heating Duration (hrs)	Cooling Method
1	4	Air cooled
2	4	Air cooled
3	4	Quenched
1	1	Air cooled
2	1	Air cooled
3	1	Air cooled
1	2	Air cooled
2	2	Air cooled
3	2	Air cooled

3.6: Load Test Setup

Eleven total beams were load tested; this included two control beams and nine cooked beams (see Table 3.5). A custom load frame was designed and fabricated, as shown in Figures 3.15 through 3.17, to provide the required resistance for the load actuator. It was essentially a large portal frame that was pieced together using available steel sections and bolted to the slab using anchors with internal threads and all-thread rod. Casters were provided at the base of the load frame to allow the frame to be rolled in and out of position as required during testing. The actuator used was a MTS single ended actuator that had a stroke of 42 inches and a compressive force capacity of 146 kips.

Figure 3.15: North elevation of test setup

Figure 3.16: East elevation of test setup

Figure 3.17: Photo of Northeast view of load test setup

3.7: Loading procedure

Before each load test, three concrete cylinders cast of the same concrete mix as the experimental beams were tested to find the compressive strength and the modulus of elasticity. Since an estimate of the compressive strength of the cylinders was necessary to measure the moduli, one cylinder from each set was loaded to failure according to ASTM C 39 (*Standard Test Method for Compressive Strength of Cylindrical Concrete*

Specimens), as shown in Figure 3.18. Once the compressive strength was known, the cylinders could be loaded to stresses slightly larger than $0.4f'_c$ while measuring the modulus of elasticity, as shown in Figure 3.19, according to ASTM C 469 (*Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*).

Figure 3.18: Compressive failure of concrete cylinder

Figure 3.19: Measurement of the modulus of elasticity

Each beam test was performed in cycles that began with force control and then changed to deflection control when appropriate (see Table 3.5). The load history was determined after experimenting with the two control beams in order to evaluate strength and stiffness degradation characteristics of the beams. Secondary cycles, or “b” cycles, were run through cycle eight in order to measure stiffness degradation. They were stopped after this point because the degradation became negligible.

Table 3.5: Typical load history

Cycle	Force (k)	Cycle	Deflection (in)	Cycle	Deflection (in)
1a, b	1	6a, b	0.50	11a	1.75
2a, b	2	7a, b	0.75	12b	2.00
3a, b	4	8a, b	1.00	13a	2.25
4a, b	8	9a	1.25	---	---
5a, b	12	10a	1.50	---	---

CHAPTER FOUR

EXPERIMENTAL HEATING RESULTS

4.1: Visual Analysis

It was not feasible to monitor the cracking of the concrete at different temperatures or durations inside the burner. Therefore, all visual observations that addressed the face of the beam being directly exposed to heat were noted after the complete heating regimen had occurred, and the burner was clear of the beam.

4.2: Color change

The intensity of the color changes increased with the duration of the burns. One and two-hour burns had no visible effect on the original color of the concrete. However, the three and four-hour burns presented with a darker grey color. In several cases, the concrete became pink along the edges where the burner was, as shown in Figure 4.1.

Figure 4.1: Color changes at B0iv-4

4.3: Cracking

The thermal cracking patterns remained consistent throughout all of the burns. Three basic types of cracks were observed. In order to simplify referring to these cracks later, the three types will be named Type 1, Type 2, and Type 3, respectively. The first pattern could best be described using a phrase typically reserved for asphalt pavements called “alligator cracking”. This was a random assortment of cracks that formed various sizes of squares on the surface of what was in this case concrete, as shown in Figure 4.2. In some cases the longer duration burns produced cracks widths that were slightly larger and more apparent, but this was not always the case.

Figure 4.2: Type 1 cracking on B3-4

The second type of thermal cracking, stemmed horizontally or diagonally from the edge of the burner and ran outward. These were the only cracks that could be observed while the beam was cooking and usually formed between 45 minutes and 1 hour from the start of heat loading. Although these cracks always formed, there was no obvious pattern to the location and amount of them. Typically, calcium carbonate (CaCO_3) was found around the cracks.

Figure 4.3: Type 2 cracking on B1-1 with moisture present

The third type was a set of cracks that were located on the lower portion of the two sides and on the bottom of the beam. First, there was a horizontal crack that ran the depth of the burner and was found approximately six inches up from the bottom. Second was a longitudinal crack found on the bottom face of the beam. The length of this crack varied between approximately 12 and 36 inches, but was usually centered with respect to the center of the burner. Finally, one or two vertical cracks would extend between these two longitudinal lines. If two cracks were extending downward, they would be located at third points with respect to the depth of the burner. If there was one, it was centered within the burn area.

Figure 4.4: Type 3 cracking on B0vi-4

4.4: Spalling

Explosive spalling occurred between 68 and 90 minutes during the first four burns on B0, with the exception of B0iii-4 where there was a second spall at 105 minutes. This contradicts Hertz (2002), who stated that explosive spalling usually occurred within the first 20 minutes if it was going to happen. Although all of the spalling locations were at the bottom corners of the beam, the characteristics do not match with the description of “corner spalling”. All examples were clearly explosive, as the sound could best be compared to a gun shot.

4.5: Moisture Expulsion

As expected, moisture was expelled from the beam while it was being cooked. It exited in two observed locations: Type 2 cracks and prestressing steel. The most notable amount of water exited the beam through Type 2 cracking. Steady streams ran down the beams and would puddle on the slab below. There was enough pressure inside the beam that air bubbles could be seen and heard being forced out with the water.

Similar observations were made where the water was forced out at the end of the beam through and around the area where the prestressing steel terminated. Air bubbles could be seen and heard being forced out the ends of the steel, and in two cases small streams of water were shot approximately 1 foot out of the beam end.

4.6: Comparison of Visual Observations

When comparing all of the heated beams by way of visual analysis, they did not differ by heating duration but by heating date. The beam used to measure temperature profiles was tested approximately 1 month before the other nine heated beams. The most notable difference between these two groups was that B0 exhibited spalling on all interior burn areas. In contrast, while the other nine beams were all heated at the center, not one endured spalling. Water was also found to exit the prestressing steel at later times when internal burn areas were being tested, as shown in Table 4.2; the cause for this could be associated with spalling.

Similarities between B0 and the other heated beams did exist when the two external burn areas were observed. As shown in Table 4.2, these two tests did not exhibit spalling and showed Type 2 cracking and moisture at the prestressing steel at comparable times.

Table 4.1: Summary of typical visual characteristics during a burn and the times they occurred (values shown in table are time in minutes)

Burn	Type 2 cracking/weeping	Spalling	*Water exits prestressing steel
B0i-3	50	75	200
B0ii-3	80	68	100
B0iii-4	105	90 & 105	120
B0iv-4	53	72 & 88	95
B0v-4	61	N/A	58
B0vi-4	43	N/A	52
B1-1	55	N/A	55
B2-1	67	N/A	67
B3-1	72	56 (cylinder)	72
B1-2	90	N/A	60
B2-2	65	N/A	80
B3-2	73	N/A	73
B1-4	57	N/A	83
B2-4	58	N/A	58
B3-4	54	N/A	54

*Water expelled at the ends of the beam where the prestress strands are terminated.

4.7: Temperature Data

The burner temperatures were somewhat erratic for B0i-3 as a result of refining the operation of the burner (see Figure 4.5). Although there was preliminary practice of burner operation, it was not feasible to rehearse the operation of the burner while installed on a beam. When the beam was inserted in the opening, unanticipated air flow problems were experienced causing the burner to act much differently than expected.

Therefore, there was a “learning curve” associated with the operation of the burner. This primarily entailed understanding the effects of propane pressure adjustments on temperature. The front door of the burner was opened at multiple periods throughout the test in order to control the effects of extreme adjustments in pressure; this explains the sharp downward spikes in the graph. Air flow modifications from the blowers were also made in the early stages of the test; however, the air flow was eventually kept constant since changes in propane pressure could accurately control temperatures.

One other factor that was not discovered immediately was the behavior of the flames from the blowers. While observing them, it appeared as though the upper blower was not lit. After multiple relighting attempts, it was assumed that there was a problem that would have to be diagnosed after the test. Therefore, the propane was shut off to this blower. After approximately 160 minutes, it was realized that the air flow may also be limiting the flames in the upper portion of the burner. The propane was then turned back on and an immediate increase in temperature was noted.

Spalling occurred at the lower, right corner of the beam after 75 minutes. The temperature of 4c then began to increase at a faster rate due to the loss of concrete cover. With the exception of thermocouple 4c, temperature readings continued to increase after the burner was shut off.

Several thermocouples did not appear to be collecting accurate data; these included 1b, 1c, and 3a. The reason for this is unknown, but was suspected to be connection issues

since 3a would fluctuate up to temperatures that were expected and then fall back down to where approximately 77°F (25°C) where 1b and 1c remained. A list of inoperable thermocouples or those that were affected by spalling are listed in Table 4.2.

Figure 4.5: Temperature history for B0i-3

Burns B0ii-3, B0iii-4, and B0iv-4 all behaved similarly to B0i-3, as shown in Figures 4.6 through 4.8. Spalling occurred at the bottom, right corner of B0ii-3 and B0iv-4 and at the bottom, left corner of B0iii-4. The spalling from B0iv-4 was large enough that it severed thermocouples 4b and 4c. It is likely this occurred because there was slack in the thermocouple wires when the concrete was being placed and the concrete pushed the wires down closer to the bottom of the beam than where the ends were attached. Temperatures continued to increase after the burner was shut off for approximately 30

minutes. There were no noticeable changes in cooling temperatures when air cooling and water quenching were compared.

Figure 4.6: Temperature history for B0ii-3

Figure 4.7: Temperature history for B0iii-4

Figure 4.8: Temperature history for B0iv-4

Burns B0v-4 and B0vi-4 exhibited no spalling. However, water began exiting the prestressing steel at a much earlier time, as shown in Table 4.1. Adjacent burn areas where spalling had occurred also became moist, as shown in Figure 4.9.

A new occurrence was the slipping of the prestressing strands, as shown in Figure 4.10. As these two regions were heated, the strands slipped into the concrete at a slow, steady rate up to 1/4 and 1/8 inches for B0v-4 and B0vi-4, respectively.

The cracking patterns remained typical. Type 2 cracking appeared to be limited in length on the side of the burn area closest to the end of the beam, as shown in Figure 4.11.

Figure 4.9: Cracking pattern for B0v-4

Figure 4.10: Moisture in B0iii-4 spalling area caused by B0v-4

Figure 4.11: View of beam end; prestressing strands slip into beam during B0v-4

Figure 4.12: Temperature history for B0v-4

Figure 4.13: Temperature history for B0vi-4

As shown in Figure 4.14, the average temperatures for all of the embedded thermocouples were calculated in order to visualize the overall trends due to heat penetration; the average burner temperature was also added to show that the burn areas were being subjected to the target temperatures. These averages only included “applicable” thermocouple readings, however. Readings were not considered applicable if the thermocouple was not reading correctly or if spalling had occurred in a spot adjacent to a thermocouple position. Table 3.1 shows a list of applicable thermocouples for each test.

Table 4.2: Applicable thermocouples

Burn	Thermocouple										
i	1a			2a	2b	2c		3b	3c	4a	4b
ii	1a	1b	1c	2a	2b	2c	3a	3b	3c	4a	4b
iii	1a			2a	2b	2c	3a	3b	3c		4b 4c
iv	1a		1c	2a	2b	2c	3a	3b	3c	4a	
v	1a		1c		2b	2c	3a	3b	3c	4a	4b 4c
vi	1a	1b	1c	2a	2b	2c	3a	3b	3c	4a	4b 4c

The resulting relationship between temperature and duration were as expected. It stands to reason that the longer concrete is cooked, the farther the heat will penetrate into the interior of the beam. In addition, the rate of temperature change will be dependent on the clear cover at a given position. This idea is better illustrated in Figure 4.15, where average temperatures were calculated for each row. It is evident that the temperatures of row 4 increase at a faster rate and that these rates decrease as the distance from the bottom of the beam increases.

Figure 4.14: Average ramping and heating phase temperatures for B0

Figure 4.15: Average row temperature histories

Figures were also desired that would indicate that the burner and KAOboard™ insulation representing a slab were functioning correctly. In this case, in order to function correctly the burner would need to maintain the desired temperatures on all sides of the beam and the insulation should slow the heat penetration to the concrete surface where the “slab” was present. As shown in Figure 4.16, deviations between the average temperatures in columns A, B, and C are negligible. If the burner was generating more heat on one side, column A or C should have had higher average temperatures that would not correlate so closely with the other columns. Average row temperatures, as shown in Figure 4.15, also show expected trends. Row 4 had the highest average temperatures which correlated with the lowest average clear covers.

Figure 4.16: Average column temperature histories

The remaining nine cooked beams, whose temperature histories are shown in Figures 4.17 through 4.25, all had similar results. Typically, Type 2 cracking would become evident at about the same time as water began to weep out of the prestressing steel at the ends of the beam. Type 1 and 3 cracking were also present after all heating regimens, but varied slightly in severity.

The 2-hour burns were tested last. As shown in Figures 4.20 through 4.22, the temperatures were lower than desired in the heating phase. This was likely due to a problem associated with the propane tank or hoses; up to twice the typical pressure of propane was pumped into the system and the temperature changes were negligible.

It should be noted that the “top” and “side” curves present in some of the graphs are in reference to the thermocouples placed under the top and side KAOboardTM. This was performed in order to measure the surface temperatures necessary for generating the temperature profiles.

Figure 4.17: Temperature history for B1-1

Figure 4.18: Temperature history for B2-1

Figure 4.19: Temperature history for B3-1

Figure 4.20: Temperature history for B1-2

Figure 4.21: Temperature history for B2-2

Figure 4.22: Temperature history for B3-2

Figure 4.23: Temperature history for B1-4

Figure 4.24: Temperature history for B2-4

Figure 4.25: Temperature history for B3-4

Figure 4.26: Average temperature histories for load tested beams

CHAPTER FIVE

DESIGN OF CONSTITUENT MATERIAL MODEL

5.1: Temperature Profile

The temperature profiles used in XTRACT were generated by using the average temperatures shown in Figure 4.26 at the desired time. The maximum average burner temperature found within the desired time frame was used to depict the exposed surface temperature. For example, the exposed surface temperature for the 2-hour burn was found by determining the maximum average temperature within the total time frame of 180 minutes. The reason for this was that the burner temperatures were found to fluctuate enough that temperature averages could be thrown off at specific small time steps. Since the actual temperature of the concrete was not undergoing these drastic fluctuations, the maximum average temperature was deemed suitable in depicting the surface temperatures.

As mentioned earlier, thermocouples were also added under the KAOboards of beams B1-2, B2-2, B3-2, B1-1, and B2-1 in order to gather surface temperature data necessary for the temperature profiles. However, as noted in the results from the heating data, the burner could not create enough heat to maintain the desired temperature in the heating phase of B3-2. Since readings were required for the 4-hour burns, trend lines were generated based on the average temperatures, excluding B3-2, under the top and side

KAOboards. These were extended to the end of the heating phase of the 4-hour burn, as shown in Figure 4.1.

Figure 5.1: Trendlines used to estimate surface temperatures under KAOboards

Preliminary cross-sectional temperature profiles were then created using the experimental data. Although these profiles would have sufficed, they were not completely symmetrical, likely due to the heterogeneous composition of concrete. Therefore, temperature averages were modified in order to more accurately depict the results of an even, three-sided burn. The actual and final modified data can be seen in Figures 4.2-4.9, along with the temperature profiles. It should be noted that intervals for the temperature gradients are at 392, 752, and 1112°F (200, 400, and 600°C).

Table 5.1: Experimental 1-hour burn temperatures

Temperature (°F)					
	Left	A	B	C	Right
Top	398	241	241	241	398
1	398	236	238	194	398
2	1133	244	224	282	1133
3	1133	237	231	282	1133
4	1133	320	327	327	1133
Bottom	1133	1133	1133	1133	1133

Table 5.2: Modified 1-hour burn temperatures

Temperature (°F)					
	Left	A	B	C	Right
Top	397	241	241	241	397
1	397	237	237	237	397
2	1134	284	225	284	1134
3	1134	284	230	284	1134
4	1134	327	327	327	1134
Bottom	1134	1134	1134	1134	1134

Figure 5.2: 1-hour burn temperature profile (°F)

Table 5.3: Experimental 2-hour burn temperatures

Temperature (°F)					
	Left	A	B	C	Right
Top	470	369	369	369	470
1	470	305	306	253	470
2	1177	306	263	370	1177
3	1177	310	293	381	1177
4	1177	470	470	469	1177
Bottom	1177	1177	1177	1177	1177

Table 5.4: Modified 2-hour burn temperatures

Temperature (°F)					
	Left	A	B	C	Right
Top	469	369	369	369	469
1	469	306	306	306	469
2	1177	370	262	370	1177
3	1177	381	293	381	1177
4	1177	469	469	469	1177
Bottom	1177	1177	1177	1177	1177

Figure 5.3: 2-hour burn temperature profile (°F)

Table 5.5: Experimental 4-hour burn temperatures

Temperature (°F)					
	Left	A	B	C	Right
Top	813	544	544	544	813
1	813	427	445	285	813
2	1228	426	388	531	1228
3	1228	499	463	573	1228
4	1228	692	697	678	1228
Bottom	1228	1228	1228	1228	1228

Table 5.6: Modified 4-hour burn temperatures

Temperature (°F)					
	Left	A	B	C	Right
Top	813	543	543	543	813
1	813	428	446	428	813
2	1229	531	388	531	1229
3	1229	574	464	574	1229
4	1229	693	696	693	1229
Bottom	1229	1229	1229	1229	1229

Figure 5.4: 4-hour burn temperature profile (°F)

5.2: Concrete Properties

The concrete stress-strain curves were generated based on a combination of equations for theoretical stress-strain models by Kent and Park (1971), previous research performed by Lie et al. (1986), and the actual compressive strengths (f_c) of the control cylinders. This combination was necessary because the compressive strengths of the beams were larger than reported in Lie et al. (1986). Thus, the ratio of the actual compressive strength of the control beams to the compressive strength reported in the Lie et al. research was calculated in order to find f_c for the desired concrete temperatures. The stress-strain

equations are shown below as Equations 5.1 and 5.2, the variables used are in Table 5.1, and the final stress-strain curves are shown in Figure 5.5.

$$\text{If } \varepsilon_c < k\varepsilon_0, \text{ then } f_c = kf'_c \left[\frac{2\varepsilon_c}{\varepsilon_0 k} - \left(\frac{\varepsilon_c}{\varepsilon_0 k} \right)^2 \right] \quad (5.1)$$

$$\text{If } \varepsilon_c > k\varepsilon_0, \text{ then } f_c = kf'_c \left[1 - Z_m (\varepsilon_c - \varepsilon_0 k) \right] \geq 0.2kf'_c \quad (5.2)$$

f_c = compressive stress at ε_c

f'_c = maximum compressive stress

ε_0 = strain at maximum compressive stress

ε_c = strain in concrete

Z_m = stress declining ratio after peak stress

$k = 1.0$ for unconfined concrete

Table 5.7: Variables used in Kent and Park's (1971) theoretical equations

Concrete Temperature °F (°C)	f_{c0} (psi)	ε_0 (in/in)	Z_m (psi)
68 (20)	8350	.002	15000
392 (200)	8350	.007	5000
752 (400)	8350	.013	850
1112 (600)	5327	.022	200

Figure 5.5: Final stress-strain curves for concrete

5.3: Prestress Loss

The equations for estimating prestress loss were used to determine the stress in the bottom strands.

$$f_{se} = f_{pi} - TL \quad (5.3)$$

$$TL = ES + CR + SH + RE \quad (5.4)$$

where:

TL = total loss

ES = elastic shortening

CR = creep of concrete

SH = shrinkage of concrete

$$ES = \frac{K_{es} E_{ps} f_{cir}}{E_{ci}} \quad (5.5)$$

where:

$K_{es} = 1.0$ for pretensioned members

E_{ps} = modulus of elasticity of prestressing tendons

E_{ci} = modulus of elasticity of concrete at time prestress is applied

f_{cir} = net compressive stress in concrete at center of gravity of prestressing force immediately after the prestress has been applied to the concrete

$$f_{cir} = K_{cir} \left(\frac{P_i}{A_g} + \frac{P_i e^2}{I_g} \right) - \frac{M_g e}{I_g} \quad (5.6)$$

where:

$K_{cir} = .9$ for pretensioned members

P_i = initial prestress force

E = eccentricity of center of gravity of tendons with respect to center of gravity of concrete at the cross section considered

A_g = area of gross concrete section at the cross section considered

I_g = moment of inertia of gross concrete section at the cross section considered

M_g = bending moment due to dead weight of prestressed member and any other permanent loads in place at time of prestressing

$$CR = K_{cr} \frac{E_{ps}}{E_c} (f_{cir} - f_{cds}) \quad (5.7)$$

where:

$K_{cr} = 2.0$ for normal weight concrete

f_{cds} = stress in concrete at center of gravity of prestressing force due to all superimposed permanent dead loads that are applied to the member after it has been prestressed

E_c = modulus of elasticity of concrete at 28 days

$$f_{cds} = \frac{M_{sd}e}{I_g} \quad (5.8)$$

where:

M_{sd} = moment due to all superimposed
permanent dead and sustained loads
applied after prestressing

$$SH = (8.2 \times 10^{-6}) K_{sh} E_{ps} (1 - .06 V/S) (100 - R.H.) \quad (5.9)$$

where:

$K_{sh} = 1.0$ for pretensioned members

V/S = volume-to-surface ratio

$R.H.$ = average ambient relative humidity

$$RE = [K_{re} - J(SH + CR + ES)]C \quad (5.10)$$

where:

$K_{re} = 20,000$ for 270 Grade stress-relieved
strand

$J = .15$ for 270 Grade stress-relieved strand

$C = 1.0$ for $f_{pi}/f_{pu} = .7$

Example calculation of prestress loss due to shrinkage of concrete (SH) for an 18 feet long 12 in x 20 in rectangular cross-section:

$$SH = (8.2 \times 10^{-6}) K_{sh} E_{ps} (1 - .06 V/S) (100 - R.H.) \quad (5.9)$$

$K_{sh} = 1.0$ for pretensioned members

$E_{ps} = 29 \times 10^6$ psi

$$V / S = \frac{(18\text{ft})(12\text{in})(12\text{in})(20\text{in})}{(2)(18\text{ft})(12\text{in})(20\text{in} + 12\text{in})}$$

$$V / S = \frac{51840\text{in}^3}{12824\text{in}^2}$$

$$V / S = 3.75\text{in}$$

$RH = 70$ (PCI Figure 3.10.12 provides a relative humidity map)

$$SH = (8.2 \times 10^{-6})(1.0)(29 \times 10^6 \text{ psi})[1 - (.06)(3.75 \text{ in})](100 - 70)$$

$$SH = 5529 \text{ psi}$$

All other calculations were performed as in the example. The final total prestress loss was 31585 psi which resulted in a final tension of approximately 24 kips/strand.

5.4: Prestressing Steel Properties

The temperature at which the residual strength of prestressing steel begins to degrade is dependent on the article that is cited; Holmes et al. (1982) indicated that it was at 392°F (200°C), while Neves et al. (1996) suggested that minor degradation began at 392°F (200°C), but no drastic changes occurred until 752°F (400°C). Therefore, both theories were modeled in XTRACT in order to compare them to the experimental results. A comparison of the reduction percentages is shown in Table 5.8.

Table 5.8: Comparison of residual strength degradation of prestressing steel

Heating duration (hours)	Prestressing steel temperature °F (°C)	Holmes et al. (1982) % Reduction	Neves et al. (1996) % Reduction
1	327 (164)	0	2
2	470 (243)	5	2
4	693 (367)	20	4

The stress-strain curves were modeled using XTRACT's theoretical equations shown below as Equations 5.11 and 5.12. The unheated properties were modeled after the stress-strain curve generated by Harmathy et al. (1970) for 68°F (20°C); the stress and strain properties used to do so are shown in Table 5.9. The yielding and ultimate stresses were then reduced by the percentages shown in Table 5.8 in order to generate the final stress-strain relationships shown in Figures 5.6 and 5.7.

$$\text{If } \varepsilon < \varepsilon_y, \text{ then } f_s = E\varepsilon \quad (5.11)$$

$$\text{If } \varepsilon < \varepsilon_{su}, \text{ then } f_s = f_u - (f_u - f_y) \left(\frac{\varepsilon_{sp} - \varepsilon}{\varepsilon_{sp} - \varepsilon_{su}} \right)^2 \quad (5.12)$$

where:

$$E = 29000 \text{ ksi}$$

f_s = tensile stress at ε

f_y = yielding stress

f_u = ultimate stress

ε_y = strain at yielding stress

ε_{sp} = strain at ultimate stress

ε_{su} = failure strain

ε = strain in steel

Table 5.9: Unheated prestressing steel properties

Material Property	Unheated Prestressing Steel
f_v	232 ksi
f_u	270 ksi
ε_y	0.008 in/in
ε_{sp}	0.05 in/in
ε_{su}	0.094 in/in

Figure 5.6: Stress-strain curves using reductions based on research by Holmes et al. (1982)

Figure 5.7: Stress-strain curves using reductions based on research by Neves et al. (1996)

Once all of the temperature profiles were generated and the material properties were organized, the XTRACT models could be created; Figures 5.8 through 5.11 show the 1-hour 2-hour, and 4-hour burn XTRACT cross-sections, while Figures 5.12 and 5.13 show the moment-curvature graphs from these final models. As predicted, the residual strength of the prestressing steel played a significant role in the moment-curvature relationships. Tables 5.10 and 5.11 show the change in elastic and inelastic EI values. Neves (1996) is shown to have equal or smaller elastic slopes, but larger inelastic slopes than Holmes (1982).

Figure 5.8: XTRACT Control model

Figure 5.9: XTRACT 1-hour burn model (see Figure 5.2 for boundary dimensions)

Figure 5.10: XTRACT 2-hour burn model (see Figure 5.3 for boundary dimensions)

Figure 5.11: XTRACT 4-hour burn model (see Figure 5.4 for boundary dimensions)

Figure 5.12: Moment-curvature graph based on research by Holmes et al. (1982)

Figure 5.13: Moment-curvature graph based on research by Neves et al. (1996)

Table 5.10: Comparison of elastic and inelastic EI

Name	Holmes (1982)		Neves (1996)	
	Elastic EI (k-in ²)	Inelastic EI (k-in ²)	Elastic EI (k-in ²)	Inelastic EI (k-in ²)
Control	34,830,000	305,000	34,830,000	306,000
1 hr	17,420,000	284,000	16,670,000	312,500
2hr	6,429,000	353,000	6,429,000	362,900
4hr	10,530,000	278,000	9,150,000	391,700

Table 5.11: Normalized difference between Holmes (1982) and Neves (1996)

Normalized Difference				
Name	Holmes (1982)		Neves (1996)	
	Elastic EI	Inelastic EI	Elastic EI	Inelastic EI
Control	1.00	1.00	1.00	1.00
1 hr	1.00	1.00	0.96	1.10
2hr	1.00	1.00	1.00	1.03
4hr	1.00	1.00	0.87	1.41

CHAPTER SIX

EXPERIMENTAL LOAD TEST RESULTS

This chapter will provide the average compressive strengths for the concrete cylinders, as well as the force-deflection, peak force-deflection, moment-curvature, peak moment-curvature, and peak stiffnesses for each tested beam. These results will then be averaged with respect to the heating duration sets to which they were subjected.

The experimentally-measured moment-curvature relationships were generated first by solving for EI from the elastic beam equation used to calculate the deflection of a simply supported beam with a concentrated load applied at mid-span. Then, the applied moment was divided by EI to compute the curvature of the beam. Although it can be seen that the beams were still picking up some load, the tests were terminated at 2¹/₄ inches because the crack sizes were becoming too large and the deflections were already beyond the practical range. See Equations 6.1 and 6.2.

$$\Delta = \frac{PL^3}{48EI} \quad (6.1)$$

where:

Δ = deflection at mid-span
 P = concentrated load
 L = span length (pin-to-pin)
 E = modulus of elasticity
 I = moment of inertia

solving for EI:

$$EI = \frac{PL^3}{48\Delta} \quad (6.2)$$

Using the elastic relationship given as EI equal to the applied moment, M , divided by beam curvature, ϕ , as shown in Equation 6.3, and setting Equations 6.2 and 6.3 equal, beam curvature can be computed, as shown in Equation 6.4.

$$EI = \frac{M}{\phi} \quad (6.3)$$

$$\phi = \frac{48M(EI)}{PL^3} \quad (6.4)$$

Substituting the relationship for moment, M , for a simply supported beam with a concentrated load at mid-span, beam a relationship for beam curvature, using the experimentally-measured applied load and deflection, is given as shown in Equation 6.5.

$$M = \frac{PL}{4} \quad (6.4)$$

$$\phi = \frac{12\Delta}{L^2} \quad (6.5)$$

Peak flexural stiffnesses, k , were calculated by dividing the peak force by the peak deflection of a given load cycle, as shown in Equation 6.6.

$$k = \frac{P \text{ (at peak)}}{\Delta \text{ (at peak)}} \quad (6.6)$$

6.1: Compressive Strength and Elastic Moduli

The results of the compressive strengths, as shown in Table 6.1, were not as expected. As opposed to having relative compressive strengths of approximately 30 percent for all three durations as found in Caple's (2007) research, the strengths did not fall below 50

percent. In addition, the cylinders representing the 2-hour burn had compressive strengths that were more comparable to the control cylinders than to the other heated specimens. It should be noted that relatively high standard deviations were calculated and could be a notable source of error.

Table 6.1: Compressive strengths of cylinders

Cylinders					Average Strengths and Deviation			
Beam	1 (psi)	2 (psi)	3 (psi)	4 (psi)	Average (psi)	Group Average (psi)	Standard Deviation (psi)	Normalized Average
Control	7335	8994	8169	8908	8352	8352	772	1.00
B1-1	4754	3713	5083		4517	5887	1335	0.70
B2-1	7627	7433	6336		7132			
B3-1	N/A	6307	5720		6014			
B1-2	7560	8231	7620		7804			
B2-2	6924	5062	5961		5982	7559	1376	0.91
B3-2	8570	8954	9150		8891			
B1-4	3677	4371	5281		4443			
B2-4	4740	5570	4742		5017	4351	802	0.52
B3-4	3606	3976	3196		3593			

The elastic moduli were calculated in accordance with ASTM C 469 (see Equation 6.7).

The results followed the same trend as the compressive strengths; however, the reductions relative to the control cylinders were more extreme. See Table 6.2 for results. Similarities were found, however, when the data was compared with the research by Cruz (1966), which indicated that at temperatures of 1112°F (600°C), the moduli should be approximately 30% of the original. While the actual values were in this range, it is not

understood why the elastic moduli data agrees with past research but the compressive strengths do not.

$$(6.7) \quad E = \frac{S_2 - S_1}{\epsilon_2 - 0.00005}$$

where:

E = chord modulus of elasticity

S_2 = stress corresponding to 40% of ultimate load

S_1 = stress corresponding to a longitudinal strain of .00005in/in

ϵ_2 = longitudinal strain produced by stress S_2

Table 6.2: Normalized elastic moduli

Cylinder			Average Strengths			
Beam	1 (ksi)	2 (ksi)	Average (ksi)	Group Average (ksi)	Standard Deviation (ksi)	Normalized Average
Control	3567	3867	3717	3717	212	1.00
B1-1	725	1099	912	1341	420	0.36
B2-1	1682	1267	1474			
B3-1	1367	1907	1637			
B1-2	1950	1574	1762	1663	404	0.45
B2-2	1064	1364	1214			
B3-2	1891	2136	2013			
B1-4	606	850	728	684	128	0.18
B2-4	809	606	708			
B3-4	708	521	615			

6.2: Control beams

Since the control beams were the first to be load tested, the number of cycles and the load history was calibrated during the testing of the control beams. Although similar, the

loading history for C1 and C2 were different from the other nine beams, as shown in Tables 6.2 and 6.3.

Table 6.3: Load history for C1

Cycle	Force (k)	Cycle	Deflection (in)
1a, b, c	1	8aa, b	0.75
2a, b, c	2	9a, b	0.75
3a, b, c	4	10a, b	1.00
4a, b, c	8	11a, b	1.25
5a, b, c	12	12a, b	1.50
6a, b, c	16	13a, b	1.75
7a, b, c	20	14a, b	2.00
8a	24	---	---

Table 6.4: Load History for C2

Cycle	Force (k)	Cycle	Deflection (in)
1a, b	1	8a, b	0.25
2a, b	2	9a, b	0.375
3a, b	4	10a, b	0.50
4a, b	8	11a, b	0.625
5a, b	12	12a, b	0.75
6a, b	16	13a, b	1.00
7a, b	20	14a, b	1.25
---	---	15a	1.50
---	---	16a	1.75
---	---	17a	2.0

While preparing to load C1, the actuator was being tested to see if it was running correctly. In doing so, a problem occurred and a large load was placed on the beam. This resulted in a flexural crack that extended up both sides of the beam approximately 10 inches. A problem also occurred during the testing of C2 where the data indicated that the anchor bolts may have started to slip. After unloading the beam, some of the bolts

needed to be tightened, reaffirming that slipping likely occurred. Once the bolts were tightened down, the testing continued without any further signs or problems with the load frame.

As can be seen in the force-deflection plots for Beam C1 in Figures 6.1 and 6.2 (Figure 6.1 shows the complete cyclic response while Figure 6.2 shows the peak response), significant yielding occurs at approximately 20 kips corresponding to a deflection of 3/8 inch resulting in an approximate elastic stiffness of $20 \text{ kips} / 0.375 \text{ in} = 53 \text{ kip/inch}$. Once significant yielding occurs, stiffness degrades quickly. However, note that throughout the load test, Beam C1 continues to carry slightly more load indicating no discernable strength degradation. As discussed previously, loading was stopped when significant cracking was observed. It should be noted that a hydraulic pump interlock was triggered during this test, explaining the sudden dip in the force-deflection plot as noted in Figure 6.2.

Using the curvature relationship given by Equation 6.6, the moment-curvature relationship for Beam C1 was generated. Figures 6.3 and 6.4 show the complete cyclic moment-curvature and peak moment-curvature relationships, respectively. Figure 6.5 provides a bar graph of the measured stiffness degradation. Note that the rate of stiffness degradation decreases rapidly starting at cycle 8a corresponding to a level of load at which significant cracking is observed, and load is carried primarily through inelastic tensile elongation of the prestressing strands.

Figure 6.1: Force-deflection for C1

Figure 6.2: Peak force-deflection for C1

Figure 6.3: Moment-curvature for C1

Figure 6.4: Peak moment-curvature for C1

Figure 6.5: Peak stiffness for C1

Similarly, as can be seen in the force-deflection plots for Beam C2 in Figures 6.6 and 6.7 (Figure 6.6 shows the complete cyclic response while Figure 6.7 shows the peak response), significant yielding occurs at approximately 24 kips corresponding to a deflection of 3/8 inch resulting in an approximate elastic stiffness of $24 \text{ kips} / 0.375 \text{ in} = 64 \text{ kip/inch}$. Once significant yielding occurs, stiffness degrades quickly. However, note that throughout the load test, Beam C2 continues to carry slightly more load indicating no discernable strength degradation. As discussed previously, loading was stopped when significant cracking was observed.

Using the curvature relationship given by Equation 6.6, the moment-curvature relationship for Beam C2 was generated. Figures 6.8 and 6.9 show the complete cyclic

moment-curvature and peak moment-curvature relationships, respectively. Figure 6.10 provides a bar graph of the measured stiffness degradation. Note that the rate of stiffness degradation starting at cycle 8a corresponding to a level of load at which significant cracking is observed is less severe compared to Beam C1, and the initial stiffness of Beam C2 is relatively larger compared to Beam C1. This slight difference is a direct result of the initial flexural crack suffered by Beam C1, as discussed previously.

Control beams C1 and C2 had comparable yielding moments of approximately 1000 and 1,200 kip-inches, respectively (see Figures 6.4 and 6.9). The 20% larger yield moment carried by Beam C2 can, again, be attributed to the initial damage suffered by Beam C1 prior to testing.

Figure 6.6: Force-deflection for C2

Figure 6.7: Peak force-deflection for C2

Figure 6.8: Moment-curvature for C2

Figure 6.9: Peak moment-curvature for C2

Figure 6.10: Peak stiffness for C2

The flexural cracking patterns occurred as expected for a typical prestressed concrete beam. These included three or four cracks that started out vertically at the bottom of the member and as they increased in height, they began to curve towards the center. Typically, these individual cracks ran down the side of one face, across the bottom of the beam, and up the other face, as shown in Figures 6.11 and 6.12. The maximum crack width observed during the loading was $\frac{3}{8}$ inches for C1 and $\frac{3}{16}$ inches for C2.

Figure 6.11: Flexural cracks on C2

Figure 6.12: Flexural cracks at side and bottom of C1

6.3: 1-Hour Burn

As can be seen in the force-deflection plot for Beam B1-1 in Figure 6.13, which shows the complete cyclic response, significant yielding occurs at approximately 25 kips corresponding to a deflection of $3/4$ inch resulting in an approximate elastic stiffness of $25 \text{ kips} / 0.75 \text{ in} = 33 \text{ kip/inch}$. Beams B2-1 and B3-1 show significant yielding at approximately 25 kips and 23 kips, respectively, corresponding to deflections of $3/4$ and $5/8$ inch, resulting in an approximate elastic stiffnesses of 33.3 and 36.8 kip/inch. Force-deflections for Beams B2-1 and B3-1 are shown in Figures 6.14 and 6.15, respectively. After the point of significant yielding is reached, beams B2-1 and B3-1 continue to pick up more load at a slightly faster rate than B1-1. Once significant yielding occurs, stiffness degrades less rapidly compared to the control beams. However, during post-yield loading, all three beams continue to carry slightly more load indicating no discernable strength degradation. As discussed previously, loading was stopped when significant cracking was observed. See Figure 6.16 for the peak force-deflection relationships for the 1-hour beams.

Figure 6.13: Force-deflection for B1-1

Figure 6.14: Force-deflection for B2-1

Figure 6.15: Force-deflection for B3-1

Figure 6.16: Peak force-deflections for 1-hour beams

Using the curvature relationship given by Equation 6.6, the moment-curvature relationships for the 1-hour beams were generated. Figures 6.17 through 6.19 show the complete cyclic moment-curvatures for Beams B1-1, B2-1, and B3-1, respectively. Comparing the force-deflection and moment-curvature behavior of three beams evaluated for a 1-hour burn, the three beams (B1-1, B2-1, and B3-1) behaved similarly in regard to ultimate strength. See Figure 6.20 for a comparison of the three beams. However, considering the initial flexural stiffness, it can be seen in Figure 6.20 that B1-1 has an appreciably smaller initial stiffness than the other two beams; conversely, beyond the point of significant yielding, flexural stiffness of all three beams are similar. Figure 6.21, which summarizes the peak stiffness as a function of test cycles, illustrates the observed stiffness degradation of the three beams. Referring to Figure 6.21, it is also interesting to note the variability in initial stiffness of these beams as compared to the similar stiffness beyond the onset of significant yielding.

Figure 6.17: Moment-curvature for B1-1

Figure 6.18: Moment-curvature for B2-1

Figure 6.19: Moment-curvature for B3-1

Figure 6.20: Peak moment-curvature for 1-hour burns

Figure 6.21: Peak stiffness for 1-hour burns

Cracking from the loading was first evident after cycle 6a. For all beams load tested, the cracks due to application of force propagated from the vertical Type 3 cracks present from the application of heat. The cracking patterns are similar to the control beams except that these flexural cracks will sometimes split once they reach the horizontal line of the Type 3 cracking. Cracks that formed away from the beam's mid-span began at the bottom of the beam and then connected with Type 2 cracks, as shown in Figure 6.22. Crack widths steadily increased with increases in deflection reaching maximum crack widths of $\frac{1}{2}$ inch at a maximum deflection of 2.25 inches.

Figure 6.22: Flexural cracking of B1-3

6.4: 2-Hour Burn

As can be seen in the force-deflection plot for Beam B1-2 in Figure 6.23, which shows the complete cyclic response, significant yielding occurs at approximately 22 kips corresponding to a deflection of 3/4 inch resulting in an approximate elastic stiffness of $22 \text{ kips} / 0.75 \text{ in} = 29 \text{ kip/inch}$. Beams B2-2 and B3-2 show significant yielding at approximately 25 kips and 24 kips, respectively, corresponding to deflections of 3/4 inch and 3/4, resulting in an approximate elastic stiffnesses 33.3 kip/inch. Force-deflections for Beams B2-2 and B3-2 are shown in Figures 6.24 and 6.25, respectively. Once

significant yielding occurs, stiffness degrades less rapidly compared to the control beams. However, during post-yield loading, all three beams continue to carry slightly more load indicating no discernable strength degradation. As discussed previously, loading was stopped when significant cracking was observed. See Figure 6.26 for the peak force-deflection relationships for the 1-hour beams. As can be seen in Figure 6.26, all three of the beams tested for the 2-hour beam have very similar behavior.

Figure 6.23: Force-deflection for B1-2

Figure 6.24: Force-deflection for B2-2

Figure 6.25: Force-deflection for B3-2

Figure 6.26: Peak force-deflection for 2-hour beams

Using the curvature relationship given by Equation 6.6, the moment-curvature relationships for the 2-hour beams were generated. Figures 6.27 through 6.29 show the complete cyclic moment-curvatures for Beams B1-2, B2-2, and B3-2, respectively. Comparing the force-deflection and moment-curvature behavior of three beams evaluated for a 2-hour burn, the three beams (B1-2, B2-2, and B3-2) behaved similarly in regard to ultimate strength. See Figure 6.30 for a comparison of the three beams. However, considering the initial flexural stiffness, it can be seen in Figure 6.30 that B1-2 has an appreciably smaller initial stiffness than the other two beams; conversely, beyond the point of significant yielding, flexural stiffness of all three beams are similar. Figure 6.31, which summarizes the peak stiffness as a function of test cycles, illustrates the observed stiffness degradation of the three beams. Referring to Figure 6.31, it is also interesting to

note the variability in initial stiffness of these beams as compared to the similar stiffness beyond the onset of significant yielding.

Figure 6.27: Moment-curvature for B1-2

Figure 6.28: Moment-curvature for B2-2

Figure 6.29: Moment-curvature for B3-2

Figure 6.30: Peak moment-curvature for 1-hour burns

Figure 6.31: Peak Stiffness for 2-hour burns

The 2-hour burns had essentially the same characteristics of 1-hour burns. Cracking began after cycle 6a. Changes in the slope of the force-deflection and moment-curvature graphs still corresponded with the formation or extension of cracks. The peak stiffness reduced notably when these cracks formed. The crack widths were approximately the same size, ranging between $\frac{5}{16}$ and $\frac{9}{16}$ inches. One difference that occurred was the crack patterns. Although some of the cracks formed by stemming off existing heat cracks, flexural cracks formed approximately 2 feet from the center of the beam, similar to that of the control beams. Figure 6.32 shows a photo of typical beam damage suffered after load testing of the 2-hour burn beams.

Figure 6.32: Typical flexural cracking for a 2-hour burn (during B2-2 shown in photo)

6.5: 4-Hour Burn

As can be seen in the force-deflection plot for Beam B1-4 in Figure 6.33, which shows the complete cyclic response, significant yielding occurs at approximately 18 kips corresponding to a deflection of $3/4$ inch resulting in an approximate elastic stiffness of $18 \text{ kips} / 0.75 \text{ in} = 24 \text{ kip/inch}$. Beams B2-4 and B3-4 show significant yielding at approximately 23 kips each corresponding to deflections of $7/8$ inch each, resulting in approximate elastic stiffnesses of 26 kip/in. Force-deflections for Beams B2-4 and B3-4 are shown in Figures 6.34 and 6.35, respectively. After the point of significant yielding is reached, beams B2-4 and B3-4 continue to pick up more load at a slightly faster rate than B1-4. As can be seen in Figure 6.36, Beam B3-4 exhibits approximately 22% lower ultimate strength than that of the other two beams. Once significant yielding occurs, stiffness degrades less rapidly compared to the control beams. However, during post-yield loading, all three beams continue to carry slightly more load indicating no discernable strength degradation. As discussed previously, loading was stopped when significant cracking was observed. See Figure 6.36 for the peak force-deflection relationships for the 4-hour beams.

Figure 6.33: Force-deflection for B1-4

Figure 6.34: Force-deflection for B2-4

Figure 6.35: Force-deflection for B3-4

Figure 6.36: Peak force-deflections for 4-hour burns

Using the curvature relationship given by Equation 6.6, the moment-curvature relationships for the 4-hour beams were generated. Figures 6.37 through 6.39 show the complete cyclic moment-curvatures for Beams B1-4, B2-4, and B3-4, respectively. Comparing the force-deflection and moment-curvature behavior of three beams evaluated for a 4-hour burn, beams B2-4, and B3-4) behaved similarly in regard to ultimate strength with Beam B1-4 exhibiting slightly less strength. See Figure 6.40 for a comparison of the three beams. However, considering the initial flexural stiffness, it can be seen in Figure 6.40 that B1-4 has an appreciably larger initial stiffness than the other two beams; conversely, beyond the point of significant yielding, flexural stiffness of all three beams are similar. Figure 6.41, which summarizes the peak stiffness as a function of test cycles, illustrates the observed stiffness degradation of the three beams. Referring to Figure 6.41, it is also interesting to note the variability in initial stiffness of these beams as compared to the similar stiffness beyond the onset of significant yielding. Additionally, note that unlike the other beams tested in this study, the beams subjected to the 4-hour burn duration exhibit flexural stiffness degradation immediately as opposed to the other beams where initial flexural stiffness appears to increase during the early stages of loading.

Figure 6.37: Moment-curvature for B1-4

Figure 6.38: Moment-curvature for B2-4

Figure 6.39: Moment-curvature for B3-4

Figure 6.40: Peak moment-curvature for 4-hour burns

Figure 6.41: Peak stiffness for 4-hour burns

Cracking began after cycle 5a for the 4-hour burns, earlier than the other three sets of beams. The cracking patterns became more similar to the 1-hour burns in that all flexural cracks propagated from existing heat cracks. Maximum crack widths during loading remained in the range of $\frac{1}{2}$ inch, as shown in Figure 6.42.

Figure 6.42: $\frac{1}{2}$ inch crack shown on B3-4 during loading

6.6: Average Loading Results

For each set of time durations, including the control set, an average force-deflection and moment-curvature relationship was generated. The averaged curves were generated by taking the average of measured force-deflection coordinate at each time set of data acquisition. However, considering the problems encountered during Beam C1 testing (recall there was initial damage, and a hydraulic pump interlock) it was decided to not include Beam C2 in the averaged relationships for the control group. Thus, the averaged relationships for the control group are taken as the measured results for Beam C2. All other data was computed by averaging the three beams in each respective group.

The general trend is evident when observing the average results of the loading tests. See Figures 6.44 for the averaged force-deflection relationships. The structural properties of prestressed concrete beams degrade as the heating duration is increased. This includes larger deflections, lower yielding moments, and lower stiffness values.

The exceptions to these trends were the 2-hour burns. These force-deflection and moment-curvature relationships are virtually identical to the 1-hour burn average. In addition, these 2-hour burns were initially found to be stiffer than the 1-hour burns until a $\frac{1}{4}$ inch deflection was reached. Considering that the applied heat load for the 2-hour burns fell short of the target load, behavior of the 2-hour burn set is expected to be somewhat similar to the 1-hour burn set.

Similarly, the average peak stiffness of each of the beam test sets was computed as shown in Figure 6.45. In this Figure, plots of the averaged stiffnesses provide further evidence of the decrease in initial stiffness as duration of applied heat is increased, and that stiffnesses are relatively unaffected by heat and duration beyond significant yielding.

Figure 6.43: Average force-deflection relationships

Figure 6.44: Average peak stiffness as a function of mid-span deflection

6.7: Evaluation of Elastic and Inelastic Slopes

The moment-curvature relationships of the averaged results were approximated using bilinear curves to evaluate the changes in effective elastic and inelastic slopes (i.e., elastic and post-yield stiffness). Figure 6.45, shows the bilinear moment-curvature relationships plotted against the experimentally-measured curves. Referring to Figure 6.45, there are notable decreases in the elastic slopes as the duration of applied heat increases. For example, the elastic slope of the 4-hour burn is 58% of the elastic slope of the unaffected beams (control). Conversely, inelastic slopes are relatively unchanged regardless of duration of applied heat; an 8% increase in the inelastic stiffness of the 4-hour duration relative to the control set was noted. The similar inelastic behavior is reasonable considering that beyond the onset of significant yielding, load is resisted primarily through inelastic elongation of the prestressing strands which, as discussed previously, regains its original material properties after having cooled. Tables 6.3 and 6.4 show the yielding and moments and curvatures for the actual and bilinear curves, respectively. Although the moments are similar, the curvatures of the actual curves are larger than the bilinear curves. For example, the actual yielding curvature for a 4 hour burn is 0.00023 in/in as opposed to a bilinear curvature of 0.00009 in/in. This occurs because the actual slopes of the curves decrease gradually before reaching the onset of significant yielding. A numerical summary of the elastic and inelastic slopes of the averaged sets are provided in Table 6.5.

Figure 6.45: Average moment-curvature graph

Table 6.5: Comparison of ϕ_y and M_y for actual relationships

	Group Average		Normalized			
	ϕ_y (1/in)	M_y (k-in)	ϕ_y (1/in)	M_y (k-in)	ϕ_y (1/in)	M_y (k-in)
C1	0.00042	950	0.00012	1210	1.00	1.00
C2	0.00012	1210				
B1-1	0.00024	1200	0.00019	1167	1.61	0.96
B2-1	0.00018	1200				
B3-1	0.00016	1100				
B1-2	0.00024	1200	0.00023	1217	1.89	1.01
B2-2	0.00024	1250				
B3-2	0.0002	1200				
B1-4	0.00022	900	0.00023	1017	1.94	0.84
B2-4	0.00024	1100				
B3-4	0.00024	1050				

Table 6.6: Comparison of ϕ_y and M_y for bilinear data

			Group Average	Normalized		
	ϕ_y (1/in)	M_y (k-in)	ϕ_y (1/in)	M_y (k-in)	ϕ_y (1/in)	M_y (k-in)
C1	0.00009	1025	0.00008	1210	1.00	1.00
C2	0.00008	1210				
B1-1	0.00013	1100	0.00010	1060	1.25	0.88
B2-1	0.00008	1050				
B3-1	0.00009	1030	0.00010	1047	1.19	0.87
B1-2	0.000095	1020				
B2-2	0.00009	1050				
B3-2	0.0001	1070				
B1-4	0.00009	765	0.00009	823	1.17	0.68
B2-4	0.00009	835				
B3-4	0.0001	870				

Table 6.7: Comparison of elastic and inelastic EI from bilinear data

Beam	Group Average		Normalized			
	Elastic EI (k-in ²)	Inelastic EI (k-in ²)	Elastic EI (k-in ²)	Inelastic EI (k-in ²)	Elastic EI (k-in ²)	Inelastic EI (k-in ²)
C1	11,390,000	496,100	15,130,000	471,700	1.00	1.00
C2	15,130,000	471,700				
B1-1	8,462,000	496,400	11,010,000	559,400	0.73	1.19
B2-1	13,130,000	590,200				
B3-1	11,440,000	591,700	11,030,000	617,000	0.73	1.31
B1-2	10,740,000	679,000				
B2-2	11,670,000	608,300				
B3-2	10,700,000	562,700	8,826,000	727,800	0.58	1.54
B1-4	8,500,000	655,000				
B2-4	9,278,000	831,700				
B3-4	8,700,000	696,600				

CHAPTER SEVEN

COMPARISON OF EXPERIMENTAL AND THEORETICAL BEAM BEHAVIOR

The experimental and theoretical moment-curvature (M-c) relationships were evaluated by comparing the experimentally-measured M-c to the M-c relationships predicted by the analytical cross-sectional fiber models. The models based on research by Holmes (1982) were consistently lower than the experimental data. See Figures 7.1 and 7.2. The relative losses of strength between heating durations were also similar. However, the strength degradation pattern of the theoretical model is not precisely in line with the experimental data. The experimental data indicates that strength loss occurs due to a 1-hour and 4-hour burn with no significant changes between 1 and 2 hour durations. The theoretical model predicts a different relationship where there is no notable affect at durations less than 2 hours.

The theoretical models based on the research by Neves (1996), as shown in Figures 7.3 and 7.4, have a different degradation pattern that is more similar to the experimental results. It indicates that strength loss occurs after the same burn durations as the experimental data, but the losses are minimal, resulting in M-c relationships that more closely approximate the experimental data.

As shown in Tables 7.1 and 7.2, the elastic and inelastic slopes varied significantly between the experimental and theoretical data. The theoretical elastic EI for both models were found to be larger than the experimental EI values, with the exception of the 2-hour

burn. In contrast, the experimental inelastic EI values were larger than both theoretical models. Considering that material overstrength and strain hardening were not considered in the analytical models, this higher predicted strength is reasonable. If an overstrength factor had been applied, it would have likely allowed the models to continue picking up load and in turn increasing the inelastic slope.

The residual steel stress-strain relationships generated by Holmes (1982) were chosen to be used in the final model. The reason for this was that with the application of an overstrength factor, the experimental and theoretical relationships would become very similar. Although it could be argued that the Neves (1996) model could reasonably estimate the yielding and ultimate moments, it can be seen that the 4-hour burn already overestimates the yielding moment of the experimental data. With the addition of an overstrength factor, this model would begin to overestimate flexural capacities, which in turn could lead to hazardous designs.

Figure 7.1: Moment-curvature comparison of experimental to theoretical results based on Holmes (1982)

Figure 7.2: Bilinear moment-curvature comparison of experimental to theoretical results based on Holmes (1982)

Figure 7.3: Moment-curvature comparison of experimental to theoretical results based on Neves (1996)

Figure 7.4: Bilinear moment-curvature comparison of experimental to theoretical results based on Neves (1996)

Table 7.1: Comparison of elastic and inelastic EI

	Experimental		Holmes (1982)		Neves (1996)	
	Elastic EI (k-in ²)	Inelastic EI (k-in ²)	Elastic EI (k-in ²)	Inelastic EI (k-in ²)	Elastic EI (k-in ²)	Inelastic EI (k-in ²)
Control	15,130,000	469,900	34,830,000	306,000	34,830,000	306,000
1 hr	10,600,000	601,000	17,420,000	284,500	16,670,000	312,500
2hr	11,020,000	617,800	6,429,000	354,000	6,429,000	362,900
4hr	9,144,000	725,000	10,530,000	278,400	9,150,000	391,700

Table 7.2: Normalized elastic and inelastic EI

	Experimental		Holmes (1982)		Neves (1996)	
	Elastic EI (k-in ²)	Inelastic EI (k-in ²)	Elastic EI (k-in ²)	Inelastic EI (k-in ²)	Elastic EI (k-in ²)	Inelastic EI (k-in ²)
Control	1.00	1.00	2.30	0.65	2.30	0.65
1hr	1.00	1.00	1.64	0.47	1.57	0.52
2hr	1.00	1.00	0.58	0.57	0.58	0.59
4hr	1.00	1.00	1.15	0.38	1.00	0.54

CHAPTER EIGHT

DISCUSSION OF EXPERIMENTAL RESULTS

8.1: Heat Cracking

Type 1 cracking was due to thermal expansions. It is likely that the cement paste and aggregate began to expand on the surface. These cracks are not believed to extend to any significant depths.

Type 2 cracking was likely caused by moisture being forced into cooler regions. Hertz (2003) addressed this topic when discussing the spalling phenomenon. Hertz stated that the moisture would either evaporate through the surface or be forced further into the concrete to depths that the heat had not reached (Hertz 2003). Basically, the moisture would follow the path of least resistance. Therefore, it stands to reason that since there was only a small portion of the length of the beam being heated, the moisture may also be forced longitudinally through the beam to get outside of the burn zone. This concept was supported by observing the manner in which the cracks formed. As shown in Figures 8.1 through 8.3, the cracks always originated at the edge of the burner and began to extend outwards toward the cooler regions of the beam.

Figure 8.1: Type 2 cracks begins to form at 57 minutes on B1-4

Figure 8.2: Type 2 cracks extend at 68 minutes on B1-4

Figure 8.3: Type 2 cracks extend further at 81 minutes on B1-4

Type 3 cracking can be attributed to thermal expansion. As the beam was heated, it expanded both in-plane and out-of-plane with respect to the length of the beam, creating stress cracks as it did so. The cracks running longitudinally on the side and bottom are a result of the beam widening and lengthening out of plane. Figure 8.4 shows these typical cracks and the arrows represent the directions that the thermal stresses are theoretically pushing. The vertical cracks that run between the two longitudinal lines were likely caused by expansion of the burn area in the longitudinal direction. Figure 8.5 illustrates this idea using the same picture. These characteristics were only seen on the bottom of the beam because this was the area where temperatures were greatest, in turn causing the largest expansions.

Figure 8.4: Longitudinal cracks formed by out-of-plane expansion on Bvi-4

Figure 8.5: Vertical cracks formed by in-plane expansion on B0vi-4

Flexural cracking was found to form at Type 3 cracks. Typically, these cracks would initially start to widen and when the deflection of the beam reached $\frac{1}{2}$ inch, the cracks

would begin to extend. Additional cracks formed between and 1 and 2 feet from the center of the beam after the 1 and 2-hour duration burns. These types of cracks were not seen in the 4-hour burns, however. It is possible that the 4-hour burns create heat cracks that penetrate deeper into the beam and allow more energy dissipation. By doing so, additional crack formations are not necessary.

8.2: Spalling

When the edges of spalling locations were compared to Type 3 cracking, it appeared that they were directly related. It was assumed that these thermal cracks not only affected the surface of the beam, but also penetrated a small depth inward and weakened the corners of the beam. The excessive build-up of steam pressure created enough force to push out this weakened portion of concrete.

Since spalling only occurred on the beam used to measure temperature profiles, it is likely that curing time was the main controlling variable. The extra month may have been enough time for some of the moisture in the exterior region of the beam to hydrate. Less moisture would result in lower internal stresses and a smaller chance of spalling.

However, since B0v-4 and B0vi-4 also did not spall, so it is also assumed that the distance to the end of the beam had some effect. If the water was in fact being pushed longitudinally through the beam, which is indicated by the characteristics of Type 2 cracking and water exiting out the prestressing strands at the ends of the beam, then a

shorter distance for moisture to travel would likely allow it to exit faster and alleviate internal stresses.

8.3: Post-Heating Rise in Concrete Temperature

Most embedded thermocouple readings indicated that the temperature inside the beam continued to increase after the burner was shut down and removed from the beam. This occurred because the heat naturally dissipates to cooler regions. While the coolest region of the beam would be the surface, it is also known that the center has cooler temperatures where the heat has not yet penetrated. Thus, while heat is escaping through the surface of the beam, it is also heating the core and raising temperatures further.

Since sufficient data was required to ensure different burn areas would behave similarly at various time durations, all of the thermocouple sets were exposed to 3 or 4-hour burns. However, by doing so, the temperature increases during cooling for 1 and 2-hour burns could not be documented. In order to remain consistent, the 4-hour burns were also based on the temperatures at that precise time, not the maximum temperatures reached.

8.4: Concrete Properties

The compressive strengths for the 1-hour and 4-hour burns were found to decrease 30% and 52% respectively, relative to the control cylinders. While these reductions are less than what was expected based on Caple's (2007) research, they can be justified using the

results found by Abrams (1971). While Abrams did also determine that siliceous aggregate concrete had a residual compressive strength equal to approximately 30% of the original at 1112°F (600°C), this was found by testing cylinders that had reached this temperature throughout the entire volume. In contrast, based on the temperature profiles generated for the beam cross-sections, it is known that the approximate maximum temperature of concrete with a clear cover of 3 inches during a 1 hour burn is 327°F (164°C); likewise, during a 4-hour burn the temperature is 693°F (367°C). While the temperatures are likely to be slightly higher in the cylinder since it is smaller and heat can penetrate through the entire diameter faster, the fact will remain that the entire cylinder is not 1112°F (600°C). Therefore, just as the beams have temperature profiles with different concrete properties for different heat-affected layers, the cylinders should too. This should increase the compressive strength capacity over a cylinder with one, consistent maximum temperature.

The compressive strength of the 2-hour burn cylinders cannot be explained in this manner, however. The compressive strengths from this group grew to 91% of the controls and 21% greater than the 1-hour burns. Although research by Abrams (1971) did indicate that it was possible for the compressive strength of concrete to increase when it reached an approximate temperature range of 392-608°F (200-320°C), these tests were performed before the specimens were allowed to cool. When allowed to cool, concrete does not see an increase in unstressed residual strength at any temperature range.

While the exact reason this strength increase occurred is unknown, there are several variables that could have contributed. One explanation is that problems with burner operation were experienced during the 2-hour burns such that target heat loads were not reached (refer back to Figures 4.20 through 4.22). Although it is unlikely the slightly lower temperature were solely responsible, it is reasonable to assume that they contributed to the higher strengths. Another variable was the cooling time. While Abrams (1971) allowed the heated specimens to cool for 7 days, these cylinders were cooled for approximately 13 hours. It is reasonable to assume that this short amount of time allowed the cylinders to maintain their heated properties. Finally, it should be noted that the standard deviation of the strengths of cylinders in this burn group was 1376 psi, which is large in comparison to the actual compressive strengths. This means that there were significant fluctuations in the actual strengths among individual cylinders, which could have potentially led to unreasonable results.

The modulus of elasticity also decreased with increasing heating duration. Although it followed the same pattern as the compressive strength, the reductions were far greater. The increase in modulus of elasticity for the 2-hour burns contradicts research performed by Cruz (1966); however, the causes are theorized to be similar to those that explain the compressive strength increase.

8.5: Loading Results

The types of flexural cracking appeared to be indicative of the initial stiffness of the beams. For example, the two beams with the highest initial stiffness were the control beams. The flexural cracks that formed were typical of reinforced concrete beams, where several vertical cracks originate from the bottom of the beam and extend upwards and curve towards the center. The group of beams with the second largest initial stiffness values was the 2-hour burns. While these beams did have flexural cracks that originated from Type 3 heat cracking, cracks also formed approximately 2 feet from the center of the beam and connected with the Type 2 cracks to extend towards the center.

Although it must be understood that concrete is a highly variable material and the exact locations where cracks form cannot be deemed an appropriate analysis tool, it seemed reasonable to conclude that the additional formation of cracking directly coincided with the larger stiffness values.

In contrast, cracking was found to mostly propagate from heat cracks for 1-hour and 4-hour burns. These two groups had similar initial stiffness values; however, the stiffness of the 4-hour burns began to degrade at a faster rate, as expected.

Although the 2-hour burns had force-deflection and moment-curvature relationships that were similar to the 1-hour burns, the overall degradation pattern was reasonable based on the understanding of how concrete properties were affected by heat. Yielding curvatures

increased with heating duration, meaning that the beams were undergoing larger deflections. The yielding moments also decreased with duration, with the exception of the 2-hour burns, indicating a loss of flexural strength. Ultimate strengths were found to degrade with a trend similar to the yielding strengths.

The elastic and inelastic slopes also changed accordingly. The elastic EI dropped to approximately 73% of the control for the 1-hour and 2-hour burns and 58% for the 4-hour burn. This coincides with the larger yielding curvatures found. In contrast, the inelastic EI increased in comparison to the control values indicating that the beams were still picking up load. The reason for this is that the loads are primarily resisted by the prestressing strands after significant yielding, and since the prestressing strands should have approximated their original properties after cooling, they should all respond similarly.

CHAPTER NINE

CONCLUSIONS AND RECOMMENDATIONS

9.1: Visual Characteristics

Three general types of heat cracking were observed: Type 1, which is similar to alligator cracking in pavements, Type 2, which extended horizontally or diagonally away from the edges of the burner, and Type 3, which were located at the bottom portion of the beam where spalling would occur. Although their severity differed slightly based on the duration of the heating, the cracks occurred at all heating durations. However, it would be difficult to perform a visual analysis based on these characteristics alone.

Color changes also occurred. The areas along the beam that were inside the burner typically turned a dark grey, while the edges sometimes turned pink. While these color changes do depict the temperature ranges they were exposed to, they did not occur in all instances and also could not be solely relied on to determine the integrity of a structural member.

9.2: Trends of Structural Characteristics

With the exception of the 2-hour burn, the compressive strength and elastic modulus of elasticity will degrade with increasing heating duration. The 1-hour, 2-hour, and 4-hour

burns will see a reduction in compressive strength of 30%, 9%, and 48%, respectively. Similarly, the elastic modulus will decrease by 64%, 55%, and 82%.

Based on the bilinear models for the experimental data, the yielding moment for the 1-hour and 2-hour burns will degrade by approximately 12% relative to the control beam; the 4-hour burn will degrade by 32%. The yielding curvatures will increase with heating duration. The 1-hour, 2-hour, and 4-hour burns will see an increase of 25%, 19%, and 17%, respectively.

The elastic EI will degrade as the heating durations increase. One-hour and 2-hour burns will decrease by approximately 27%, while the 4-hour burns will decrease by 42%.

Inelastic EI values are not expected to degrade. The 1-hour, 2-hour, and 4-hour burns are expected to increase by 19%, 31%, and 54%, respectively.

9.3: Constituent Material Models

The XTRACT model based on research by Holmes (1982) shows an average underestimation of flexural strength of approximately 200 kip-in below the experimental results. In contrast, the model based on Neves (1996) generates estimates that vary between 0 and 250 kip-in, depending on the curvature of the beam. Since the model based on Holmes (1982) can be modified with an overstrength factor to account for strain hardening, the estimates would become more accurate than the Holmes model.

9.4: Recommendations for Future Research

Heating T-shaped prestressed concrete beams would likely be the next step in this research. The experimental results from this project do show that the KAOboardTM provided insulation, but it is not known to what degree of accuracy it simulated an actual slab.

Additional testing should be performed on the residual strength of prestressing strands after being heated. Although the previous research yielded data that was comparable to the experimental results, more studies are in order to ensure their accuracy. It would also be beneficial to know if there are changes in the residual strength based on the level of prestressing force found in the strands when they are heated.

Finally, it would be advantageous to core samples from heat exposed beams and compare the results of a thermogravimetric analysis to the temperature readings from embedded thermocouples. This would provide more evidence of the feasibility of using TGA to estimate temperature histories, and more importantly, cross-sectional temperature profiles.

BEAM DESIGN INFORMATION

Figure A.1: Beam detail by Tindall

Quality Control Pour Sheet

JOB NAME: **RESEARCH BEAMS**
 JOB #: **0** BED: **UNIV. A / B**
 MEMBER TYPE: **20 C 8**
 D-SHT: **D-0** POUR #: **6803**
 DESIGNED FORCE: **1/2" 27,000**
 1/2" PACK NUMBER: **910154901**
 1/2" PACK NUMBER: _____
 1/2" PACK NUMBER: _____
 1/2" PACK NUMBER: _____

STRAND PATTERN

RAM: 2666 / 2634 / 2136 INITIAL: 2706 2569 2765

STRAND MFG.: AMERICAN SPRING WIRE

TIME @ STRESSING: **10:00 AM** Q.C. INSPECTOR: **R. D. Lindler**

Anticipated Concrete Temp. _____
 Ambient Temp. @ Stressing _____
 % Of Thermal Adjustment _____

STRAND #	ACTUAL GAUGE READING	ELONGATIONS DESIRED	ACTUAL	+/- 5%	STRAND #	ACTUAL GAUGE READING	ELONGATIONS DESIRED	ACTUAL	+/- 5%
1	27900	14.70	14.70	5%	17				
2					18				
3					19				
4					20				
5					21				
6					22				
7					23				
8					24				
9					25				
10					26				
11					27				
12					28				
13					29				
14					30				
15					31				
16					32				

LEADER: **ALEX**

FOREMAN: **LINDLER**

MIX # **64**
 YARDS: **6 1/2**

POUR DATE: **2/18/2008**

PLACEMENT: _____ AM / PM
 COVER TIME: _____ AM / PM

TIME CYL. MADE: _____ SLUMP: _____ CONCRETE TEMP: _____ / TEMP

CYL. MADE BY: _____ WEATHER: CLEAR / CLOUDY / WINDY / RAIN / MIST

RELEASE: **3000** 28 DAY STRENGTH: **5000**

CALIBRATED FORCE (Circle Ram Being Used)

RAM #	2666	2634	2136
PLAT	29500	28100	27900
ELONG.	14.70	14.70	14.70
+/- 5%	0.70	0.70	0.70

 C
 O
 L
 D

PULL	FORCE	+/- 5%
30300	15.20	0.80
29600	15.30	0.80
30900	15.50	0.80
28900	15.20	0.80
29200	15.30	0.80
29400	15.50	0.80
28800	15.20	0.80
29000	15.30	0.80
29300	15.50	0.80

 W
 K
 A
 F
 H
 K
 R

Figure A.2: Pour sheet for 2/18/08

Quality Control Pour Sheet

JOB NAME: RESEARCH BEAMS STRAND PATTERN:

JOB #: 0 BED: UNIV. A / B

MEMBER TYPE: 20 C 8

D-SHT: D-0 POUR #: 6804

DESIGNED FORCE: 1/2" 27,000

1/2" PACK NUMBER: 80278301

1/2" PACK NUMBER: _____

1/2" PACK NUMBER: _____

1/2" PACK NUMBER: _____

RAM: 2666 / 2634 / 2136 INITIAL: 2706 2569 (2765) Anticipated Concrete Temp. _____
 Ambient Temp. @ Stressing _____
 % Of Thermal Adjustment _____

STRAND MFG.: AMERICAN SPRING WIRE

TIME @ STRESSING: 10:30 Q.C. INSPECTOR: Randy P.

STRAND #	ACTUAL GAUGE READING	ELONGATIONS DES. REQ.	ACTUAL	1 / - 5%	STRAND #	ACTUAL GAUGE READING	ELONGATIONS DES. REQ.	ACTUAL	1 / - 5%
1	29.000	15 3/4	15 3/4	3/4	17				
2	29.000	15 3/4	15 3/4	3/4	18				
3					19				
4					20				
5					21				
6					22				
7					23				
8					24				
9					25				
10					26				
11					27				
12					28				
13					29				
14					30				
15					31				
16					32				

LEADER: ALEX FOREMAN: LINDLER

MIX # 64 POUR DATE 2/19/2008 PLACEMENT: _____ AM / PM
 YARDS: 6 1/2 COVER TIME: _____ AM / PM

TIME CYL. MADE: _____ SLUMP: _____ CONCRETE TEMP. _____ / TEMP _____

CYL. MADE BY: _____ WEATHER: CLEAR / CLOUDY / WINDY / RAIN / MIST

RELEASE: 3000 28 DAY STRENGTH: 5000

CALIBRATED FORCE (Circle Ram Being Used)

RAM #	2666	2634	2136
BT. LI.	29500	28100	27900
ELONG.	14.70	14.70	14.70
+/- 5%	0.70	0.70	0.70

*
C
O
L
D

2000		2634		2136	
2000	2634	2000	2634	2000	2634
30300	30300	15.20	0.80		
@ 4%	30600	15.30	0.80		
@ 5%	30900	15.50	0.80		
2634 @ 3%	28000	15.20	0.80		
@ 4%	29200	15.30	0.80		
@ 5%	29400	15.50	0.80		
2136 @ 3%	28800	15.20	0.80		
@ 4%	29000	15.30	0.80		
@ 5%	29300	15.50	0.80		

W
E
A
T
H
E
R

Figure A.3: Pour sheet for 2/19/08

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