RESIDUAL AXIAL CAPACITY OF FIRE EXPOSED REINFORCED CONCRETE COLUMNS

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ABSTRACT

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Fire represents one of the most severe environmental loading conditions that a structure may experience during its design lifetime. Reinforced Concrete (RC) structural members can experience some level of damage following moderate or severe fire exposure. To ensure safe future use of the structure, and to develop adequate retrofitting measures for fire damaged concrete members, it is essential to evaluate the residual capacity of RC members following fire exposure and prior to re-occupancy.

A limited number of experimental studies and approaches exist today that accounts for the residual capacity of fire exposed RC columns. Therefore, experimental studies have been undertaken as part of this research to understand the post-fire behavior of RC columns exposed to realistic fire scenarios. Two full-scale normal strength concrete (NSC) columns have been cast and subjected to realistic restraint, loading and fire exposure conditions. The experimental studies conducted indicate that peak rebar temperatures can occur up to 100 minutes and 80 minutes after the end of the heating phase of a 90-minute and 120-minute fire exposure, respectively. The post fire residual capacity testing indicated that the RC columns can retain up to 34% of its real nominal capacity for a 90-minute exposure and up to 29% for a 120-minute exposure. However, much of the design capacity of columns is retained after exposure to fire.

In addition to the experimental studies undertaken, a numerical model developed using the commercially available finite element (FE) software, ABAQUS, is used to predict the thermal and mechanical response of RC columns before, during and after realistic fire exposure.

This thesis is dedicated to my loving wife, Kierstyn Hibner. Without your love and support, this thesis would not be possible. I would also like to dedicate this work to the memory of my late father, Robert T. Hibner (1953-2017).

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KEY TO SYMBOLS

A_c = Area of concrete

A_s = Area of longitudinal reinforcing steel

A_g = Gross cross-sectional area

β_d = Stiffness reduction factor

C_s = Temperature dependent specific heat of reinforcing steel

$c_p(\theta)$ = Temperature dependent specific heat of concrete

C_m = Factor to relate actual moment diagram to an equivalent uniform moment diagram

δ = Moment magnification factor

E_c = Elastic modulus of concrete (room temperature)

E_s = Elastic modulus of reinforcing steel (room temperature)

$E_{s,T}$ = Temperature dependent elastic modulus of reinforcing steel

e_{min} = minimum eccentricity

e = calculated eccentricity

ε_c = Strain in concrete

$\varepsilon_{c,\theta}$ = Temperature dependent strain in concrete

$\varepsilon_{cu,\theta}$ = Ultimate temperature dependent strain in concrete

ε_s = Strain in reinforcing steel

$\varepsilon_{sp,T}$ = Temperature dependent strain in steel at the proportionality limit

$\varepsilon_{st,T}$ = Temperature dependent tensile strain in steel

$\varepsilon_{c,\theta}$ = Temperature dependent strain at the instant of rupture in steel

f'c = concrete compressive strength

f_s, f_y = Room temperature yield stress of reinforcing steel

f_{sp} = Stress at proportionality limit in steel

$f_{sy,T}$, $f_{y,T}$ = Temperature dependent stress of reinforcing steel

h = Column dimension in direction of eccentricity

I_g = Gross cross-sectional moment of inertia

K = Effective length factor of column

L_u = Unbraced length of column

λ = Temperature dependent thermal conductivity of concrete

M_n = Nominal moment capacity

M_u = Ultimate moment

P = Applied axial load

P_0 = Nominal un-factored axial load

P_{cr} = Euler buckling load

P_u = Ultimate axial load

ϕ = Reduction factor

r = Radius of gyration of section in the direction of eccentricity

SG = Strain gage

$\sigma_{c,\theta}$ = Temperature dependent stress in concrete

σ_s = Stress in reinforcing steel

TC = Thermo-couple

T_{max} = Maximum temperature of fire

θ,T = Temperature

V_{20} = Pulse velocity in undamaged concrete at room temperature

V_{min} = Pulse velocity in concrete after fire exposure

w_c = Unit weight of concrete

CHAPTER 1

1 INTRODUCTION

1.1 General

Concrete is widely used as a structural material in building construction. This is primarily due to locally available constitutive materials for making concrete, cost-effectiveness and durability properties. Concrete is also a versatile material in that it can be cast into any shape for a wide range of architectural and structural uses. Structures made of conventional concretes also exhibit superior performance under fire situations.

In the design life of a structure, fire represents one of the most severe environmental loading conditions that it can experience. For this reason, structural members in buildings must satisfy fire resistance requirements as fire safety is one of the key considerations in building design [29,25,11]. However, while fires do occur in structures, historical data shows that complete collapse of a structural system due to fire is a rare event [7]. Probability of such failure in reinforced concrete (RC) structures is further lowered due to the superior thermal properties and slower degradation of mechanical properties of concrete with temperature [51]. Therefore, after most fire incidents, it is reasonable to assume that RC structures may be opened to re-occupancy with adequate repair and retrofitting.

However, the extent of damage caused by a fire in a RC structure is highly variable. Under severe fire exposure, RC members could experience significant structural damage resulting from loss of concrete due to fire induced spalling, high rebar temperatures and relatively large permanent deformations. A moderate fire exposure may not result in noticeable deformations or loss of concrete section due to spalling, and thus the loss of structural capacity may not be significant. Thus, there is an uncertainty regarding the residual capacity of fire exposed RC members due to

the extent of temperature induced degradation in material properties and the extent of recovery of capacity. Prior to re-occupancy after a fire incident, it is imperative to assess if sufficient residual capacity exists in structural members. Such an assessment also forms the basis in developing appropriate repair measures in fire damaged RC structures.

At present, there are limited approaches available for evaluating the residual capacity of fire damaged RC columns. Most of the existing approaches utilize modified room temperature strength equations, with temperature dependent strength reduction factors to account for temperature induced degradation in mechanical properties of concrete and reinforcing steel, which is based on the peak temperatures experienced during fire exposure. Current approaches do not account for realistic material properties of concrete and rebar during and after fire exposure, post-fire residual deformations, load level, and restraint conditions present during fire exposure in evaluating the residual capacity. It is therefore necessary to develop a rational approach considering realistic fire exposure scenarios, structural conditions and residual mechanical properties, in evaluating residual capacity of fire exposed RC columns.

1.2 Concrete Behavior at Elevated Temperatures

Generally, concrete provides the best fire resistance out of any structural material. This fire resistance is due to the insulative thermal properties and slower degradation of mechanical properties that NSC exhibits. The chemical combination of cement and aggregates in the presence of water yields a material that is inert and has low thermal conductivity, high heat capacity, and slower strength degradation at elevated temperatures. The slow rate of heat transfer within concrete enables the material to sustain higher mechanical properties as well as providing an effective fire shield to adjacent spaces within a structure.

However, concrete, like many other materials, loses its strength and stiffness properties when exposed to fire. At elevated temperatures, the strength properties of concrete reduce at a much slower rate than other common building materials, such as wood or steel. This means that structural members made of concrete can sustain an applied load longer than the same structural member made of a less fire-resistant material, such as steel.

Fire induced spalling is another phenomenon that concrete can experience at elevated temperatures. When the pore pressure in a heated concrete section exceeds the tensile strength of concrete, spalling of the section will occur [17]. Spalling is the breaking off of concrete layers in the structural member and can often be explosive in nature, depending on the fire and concrete characteristics [17].

Spalling is more common in concrete members made of structural High Strength Concrete (HSC), but it can still occur in NSC [38]. The spalling of concrete at elevated temperatures results in a loss of cross section and therefore a significant reduction in capacity at the instant of spalling. The spalling will also expose the reinforcing steel of the concrete member directly to the fire. Spalling is a direct result of the buildup of pore pressure within the concrete [17]. The high vapor pressure that builds up during a fire within the concrete may not be able to escape easily. When the effective pore pressure exceeds the tensile strength of concrete, spalling occurs. The process of spalling can be explosive in nature, and is dependent on the characteristics of the fire exposure, load level, and the concrete itself [27].

1.3 Behavior of RC Columns at Elevated Temperatures

Reinforced concrete (RC) columns can undergo a significant reduction in strength and stiffness during exposure to a fire, and much of the reduction is not recoverable. This is a result of the rise in temperature of both concrete and steel reinforcement, causing the material properties to degrade.

There are several factors that will influence the extent of the loss of strength of an RC column exposed to a real fire. These factors include, but are not limited to, the type and duration of fire exposure, the size and loading on the column, and the temperature levels the concrete and steel reinforcement experience during a fire.

Severe fire exposure to a RC column can cause significant structural damage to the member leading to partial or full collapse. This damage could include explosive spalling, exposure of reinforcing steel, and large permanent deformations of the member. These types of damage are indicative that the RC column has failed structurally, and may no longer be able to sustain its applied load. A moderate fire exposure could result in the RC column to exhibit little to no spalling at all, but its load carrying capacity could have been reduced. It is therefore imperative to establish a way to evaluate the residual capacity of such columns in which the extent of damage is not immediately obvious.

The current practice of evaluating the fire resistance of a structural member is to expose that member to a standard fire, such as ASTM E-119 or ISO-834, and monitor its behavior over the period of fire exposure. When the member can no longer sustain its intended design capacity, the time to that point is taken as the fire resistance of the structural member.

A typical fire resistance experiment conducted on a simply supported RC column can be seen in Figure 1.1. The depicted RC column is subjected to an axial load prior to fire exposure. Then, fire is applied to the member for a pre-determined amount of time, which follows a time-temperature curve as presented in either ASTM E-119 or ISO-834. Normally, visual observations are made during fire exposure, as well as recording of axial deformations, cross section temperatures and strains.

Figure 1.2 depicts the typical thermal and mechanical (axial deformation) response of a fire exposed RC column, which was derived from Raut, 2011 [48]. While a standard fire exposure exhibits a rapid rise in temperature over a relatively short period of time, the rebar and column center temperatures do not begin to increase until several minutes after this rapid temperature increase period. This phenomenon is owed to the insulative properties of concrete. It should also be noted that the column center and rebar temperature appear to be continuing to increase at the end of the reported test. Furthermore, there are significant axial deformations (in the form of expansion followed by contraction) throughout the duration of fire exposure in a standard fire test. It becomes obvious that temperatures and axial deformations continue to change, even after the fire exposure has ceased. A standard fire test does not account for this cooling phase, which is part of a realistic fire.

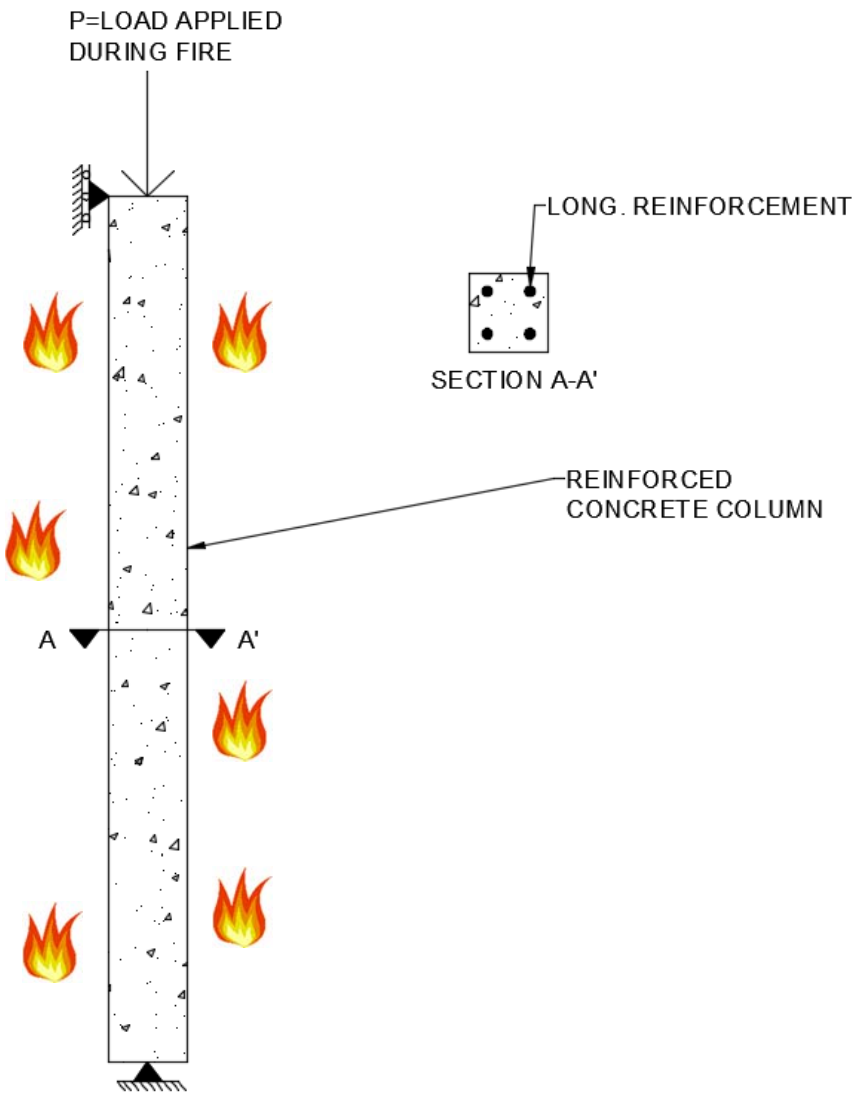

Figure 1.1. Typical Fire Resistance Experiment for RC Columns (Flame Image from Microsoft Word 2016).

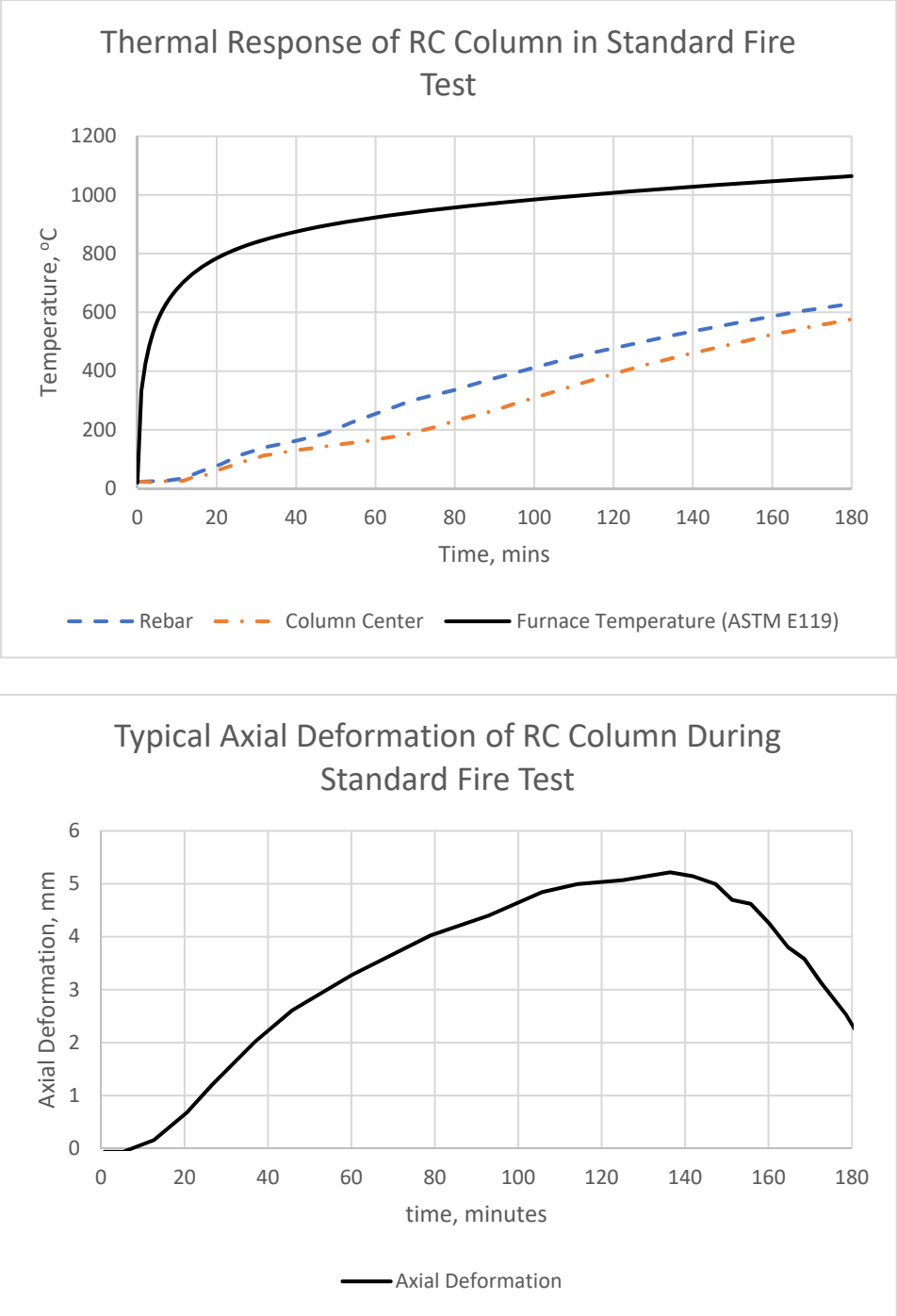

Figure 1.2. Typical Thermal (top) and Mechanical (bottom) Response of a RC Column under a Standard Fire Test. Presented Results Reported by Raut, 2011 [48].

Realistic fires in buildings can be different and have a cooling phase resulting from a lack of fuel and/or ventilation (oxygen). Recent catastrophic failures of structures shortly after a fire was extinguished, such as the parking structure that collapsed in Gretzenbach, Switzerland in 2004 [28], proves there is a need to study the cooling behavior of RC structural members after a fire exposure. Standard fire tests may not be suitable enough to fully characterize the behavior of a structural member in a realistic fire scenario. Further, there is a need to develop numerical models, which are a cheaper alternative to conducting full scale fire experiments, that accounts for the cooling phase which are typical of realistic fires. Figure 1.3 illustrates the differences between the standard fires (ASTM E-119 and ISO-834) and realistic fire exposures.

1.4 Residual Response of Fire Exposed RC Columns

The extent of damage to a RC column during a fire event is influenced by several factors, including fire severity, residual material properties of rebar and concrete, recovery time following the cooling period, temperature induced bond degradation, load level, and restraint conditions present during fire exposure. Many of these factors are interdependent and can vary significantly in different scenarios. Each of these factors that influence the residual capacity of RC columns are discussed below.

Severity of fire exposure can influence the residual capacity of a fire exposed RC column significantly. Fire exposure in a structure depends on fuel load density, ventilation characteristics, and geometrical parameters of the fire compartment [19]. These factors may evolve with the growth of the fire, for instance, a sudden increase in ventilation may occur due to the breakage of glass windows. Furthermore, presence of active fire protection systems such as sprinklers and fire fighter intervention can have significant effect on the rate of fire temperature increase, peak fire temperature as well as the time taken for the fire temperature to cool down.

Material properties of concrete vary during heating and cooling phases of fire exposure as well as upon cool down to ambient conditions. Thermo-physical, mechanical and deformation properties of concrete change substantially during the heating phase of fire exposure primarily due to the breakdown of the Calcium-Silicate-Hydrate (CSH) gel and loss of moisture present in concrete [31]. The material properties of concrete change further during the cooling phase, due to micro-cracking and chemical changes occurring during heating. Also, the Load Induced Thermal Strains (LITS, also referred to as Transient Thermal Creep, Drying Creep or Pickett's Effect [46]) occurs in concrete during the first heating under load and does not recover upon cooling of the member [6].

Post-fire residual material properties of concrete upon cooling are significantly different from properties during fire exposure. Namely, the residual compressive strength of concrete exposed to temperatures more than 220ºC can be up to 10% lower than its compressive strength at elevated temperature, for one to six weeks after fire exposure [50]. Additionally, the fire exposed concrete will recover part of its original room temperature compressive strength, with at least six months of time at room temperature [35].

Aside from temperature induced deterioration in the RC column, the loading level and restraint conditions present during fire exposure can affect the level of fire damage. Presence of higher loads (stress) and axial restraint influences the stress history experienced by the RC column during fire exposure. Consequently, these structural parameters can affect residual capacity of fire exposed RC columns.

Thus, the residual strength evaluation of fire exposed RC columns is quite complex and depends on several factors. Critical factors discussed above need to be accounted for when evaluating the residual capacity of fire exposed RC columns; which is currently lacking in literature.

1.5 Research Objectives

The above discussion infers that there are significant gaps on the fire response of RC columns under realistic fire scenarios, as well as the residual capacity of RC columns after the fire has been extinguished. To address this gap, the following research objectives were set out as a part of this Thesis:

- Undertake a state-of-the-art review on the residual capacity of RC columns subjected to fire. The review will cover experimental and numerical studies, provisions in codes and standards as well as high temperature material properties.
- Design and fabricate two RC columns for undertaking fire tests.
- Perform fire resistance tests on RC columns, and evaluate their residual capacity after fire exposure.
- Develop a finite element model for predicting the response of RC columns under realistic fire, load and failure conditions. The model will account for temperature degradation in material properties, restraint conditions and residual deformations in evaluating residual capacity.
- Validate the above model from experimental fire exposure tests on RC columns made of NSC.

1.6 Scope

This study, which has been undertaken to address the above objectives, is presented in five chapters. Chapter 1 provides a general background to the fire response of RC columns and presents the objectives of this study. Chapter 2 summarizes a state of the art review on the behavior of RC columns exposed to fire. The review includes a summary of experimental and analytical studies, as well as provisions in current codes of practice for evaluating the residual capacity of reinforced

concrete structures. Also, a review of high temperature material properties and associated constitutive relationships of concrete and reinforcing steel is presented in Chapter 2.

Chapter 3 deals with the fire resistance and residual capacity experiments conducted on two RC columns under realistic fire, and loading scenarios. Results from the fire tests as well as the residual capacity testing are used to discuss the response of the RC columns under these realistic conditions.

Chapter 4 presents the details of implementation and use of a finite element based numerical modeling of the fire response of RC columns. The validation of the finite element model is also presented within this chapter. The validation compares predictions from the developed model to the available test data from literature, as well as the fire test results presented in Chapter 3. Finally, Chapter 5 summarizes the main findings from the current study and presents recommendations for further research.

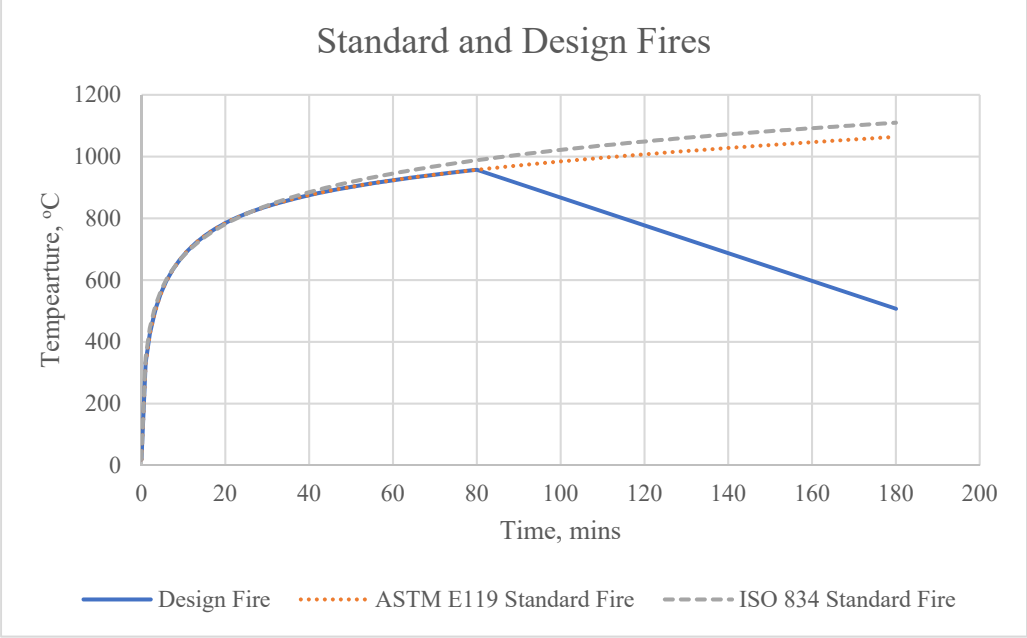

Figure 1.3. Comparison of Standard and Design Fires.

CHAPTER 2

2 STATE-OF-THE-ART REVIEW

2.1 General

Performance of concrete structures under fire exposure depends on a number of factors, including temperature dependent material properties of concrete and reinforcing steel, load level, restrain conditions and fire exposure. Current code provisions for evaluation of the fire resistance of concrete columns are based on a prescriptive methodology and often determine failure of a member (column) when the steel rebar temperature reaches a critical value. These provisions were derived from standard fire tests carried out on RC members. Since the 1970's numerous studies have been conducted to develop a better understanding on the behavior of RC columns during standard fire exposures, without due consideration to realistic fire scenarios, by specifically including a cooling phase. Moreover, researchers have focused on studying a variety of methods to determine the degree of damage that a concrete member experiences during a fire. These methods are based on local sampling of a fire damaged RC member with little consideration given to the residual behavior of a concrete member at the global level. Further, current codes of practice do not have specific approaches to evaluate residual capacity of fire damaged RC columns.

This chapter provides a state-of-the-art review on the experimental and numerical studies related to the residual capacity of RC columns subjected to fire. Furthermore, this chapter highlight's a review of the post-fire capacity provisions for RC columns provided in various codes and standards of practice.

2.2 Experimental Studies

Limited full-scale experiments were conducted in the past to evaluate the residual capacity of fire damaged RC columns. In general, the experiments involved three steps, as follows:

- Exposing concrete members (columns) to a standard fire and then cooling the specimens with either water quenching or air cooling.
- Use nondestructive testing (NDT) techniques, or direct (destructive) testing to ascertain the residual concrete strength of the specimen after fire exposure.
- Subjecting the fire exposed members to a mechanical load after cooldown to determine the actual capacity of the sample.

A summary of notable residual capacity tests on fire damaged RC columns is presented in Table 2.1. For each of the studies reviewed, the objectives, test parameters, test methods/features, test strengths/drawbacks (if any), and primary conclusions are summarized in Table 2.1. Significant findings of each of these studies are discussed in detail below.

2.2.1 Tests by Lie, Rowe, Lin (1986)

Lie, et al. [40], cast two reinforced normal strength concrete columns (305 x 305 x 3810 mm) and subjected them to an ASTM E119 standard fire with a controlled cooling phase that is specified in ISO 834. One column was exposed to a 60-minute fire duration and another column was exposed to a 120-minute fire duration. In the case of 60-minute fire exposure, cooling was achieved at 500^{o}C per hour. In the 120-minute fire exposure test, cooling was done at 250^{o}C per hour. The load level maintained during fire exposure was approximately 60% of the design ACI capacity of each column.

Lie, et. al [40], reported the residual RC column capacity after the noted fire exposures. The calculated (un-factored) nominal axial capacity of columns at room temperature was 3545 kN. The reported residual capacity of fire exposed columns was 1987 kN and 2671 kN for the 60-minute and 120-minute fire exposure, respectively. These reported values are counter-intuitive. Structural members subjected to longer fire durations should retain less capacity as compared to a shorter duration of fire exposure on a similar member. Moreover, the reported pulse velocities of the column exposed to a 120-minute fire further indicate that the concrete has experienced more damage than that of the column subjected to a 60-minute fire exposure. Lie, et. al [40] did not provide any reasoning for these contradictions and are further evidence that additional research in the area of residual capacity of RC columns after fire exposure is warranted.

Lie, et. al. [40], carried out ultrasonic pulse velocity (UPV) tests prior to fire exposure of each column, about 20 hours after the start of the fire test, and about 25 hours after the start of the fire test on only the mid-section of each column in a spalled region. The scheme of measurements taken consisted of direct transmission through the column cross section (meaning the transducer is placed on one face of the column and the receiver is placed on the opposite face of the column, in line with the transducer). Direct UPV measurements were take across the entire height of the column before and after fire testing. Delaminated material of the middle portion of the column was chipped away with hand tools and UPV measurements were also taken in this region. The mechanical sound wave transit time from the transducer is divided by the distance traveled giving a pulse velocity. Pulse velocity measurements for the column subjected to a 60-minute fire exposure were 64% lower than the original measured value prior to fire exposure. The 120-minute fire exposed column's pulse velocity was 71% lower than the original value [40]. These findings indicate that lower pulse velocities coincide with concrete that has experienced more fire damage.

The study did not report on estimation of residual capacity of RC columns after fire exposure from UPV measurements.

2.2.2 Tests by Colombo and Felicetti (2007)

Colombo and Felicetti [15] conducted nondestructive testing techniques on uniformly fire damaged concrete cubes, small concrete panels (275 x 550 x 80 mm), and on a concrete duct protecting electrical cabling in railway tunnels subjected to an ISO 834 fire. The concrete cubes were subjected to a series of thermal cycles up to T_{max} = 200, 400, 600 and 800ºC at a heating rate of 5ºC per minute. The cubes were then tested with a series of NDT methods, to determine their sensitivity to thermally induced strength loss. The NDT tests conducted on the concrete cubes were the Schmidt rebound hammer and Ultrasonic Pulse Velocity (UPV). The testing on the cubes showed a remarkable dispersion between the UPV and Schmidt hammer concrete strength results, which was due to specimen's size, experimental procedure, material porosity, and initial moisture content prior to fire exposure [15].

The concrete panels cast were intended to establish a method of determining the thermal profile through the depth of the specimen from a fire exposure up to T_{max} = 750ºC on one side of the panel, while keeping the opposite side of the panel cooled with a fan [15]. Thermocouples were used to determine the temperature gradient within the panels, and then plotted versus the residual cube strength of the previously mentioned testing of the concrete cubes. The results showed a marked concrete strength decrease corresponding to increasing temperatures observed in the profile of the panels.

Rebound hammer and UPV tests were carried out on the concrete electrical duct subjected to an ISO 834 fire with a controlled cooling phase. The rebound hammer tests indicated a decrease in concrete strength with increasing height of the specimen, which would be typical in a realistic fire

which is a result of higher temperatures at the top of the specimen. The higher temperatures at the top of the specimen are also a result of the geometry of the duct itself and the fire being applied with a vertical furnace.

In addition to the UPV tests performed on the concrete test specimens, digital camera colorimetry and drilling resistance of the fire damaged specimens were conducted on the concrete electrical duct. Concrete is expected to exhibit color changes with increasing temperatures from normal to pink or red from 300 to 600ºC, whitish grey from 600-900ºC and buff from 900 to 1000ºC [15]. Digital images were taken in the test program of the fire damaged concrete specimens and a temperature profile was estimated from the images. The strength of the color change measured is more pronounced for siliceous aggregates (which were used in the reference study) and less so for calcareous and igneous aggregates [15].

The drilling resistance technique conducted consisted of monitoring the work dissipated per unit depth being drilled with a standard battery powered hammer drill. Fire damaged material will exhibit a smaller amount of work dissipation in fire damaged concrete. The work dissipation measured increased with increasing drilling depth in fire damaged specimens, up to a constant value of work dissipation. The depth at which the constant work dissipation occurs is indicative of reaching virgin concrete. This method provides another indication of depth of fire damage in a fire exposed concrete structure.

Of the NDT methods carried out in this program, the most reliable methods are that of UPV measurements as well as the drilling resistance of fire damaged structures [15].

2.2.3 Tests by Kodur, Raut, Mao, Khaliq (2013)

Researchers of this study fabricated and tested five high strength concrete columns (203 x 203 x 3350 mm), with the specified concrete compressive strengths reported to be ranging from 62 to 91 MPa. Each specimen was subjected to a standard ASTM E-119 fire followed by a well-defined cooling phase. The objective of this study was to evaluate the residual strength of RC columns after exposure to fire. The five HSC RC columns were tested for residual capacity after the cooling phase had completed.

During each fire exposure duration for each of the tested columns, a load ratio of 40 to 60 percent of the design load carrying capacity of the column was maintained. The fire exposure zone was the middle 1700 mm of each column. Post fire residual strength tests were carried out on each specimen 24 hours after completion of the fire resistance tests. The columns were tested under the same support conditions as that during fire exposure. The columns were loaded at a rate of 10 kN per minute until failure occurred.

The primary conclusions drawn from these tests are that RC columns retain much of their original load carrying capacity (80 to 90 percent) after fire exposure, particularly if fire induced spalling does not occur. Kodur, et al. [38], also indicated that the rebar temperature exposure will have the highest influence on the residual capacity of an RC column [38]. The relatively high load carrying capacity of each column was attributed to an increase in stiffness of the RC column from the cooling of the member. Another factor that influences the residual capacity is the strain hardening of the steel that will occur during loading.

2.2.4 Summary

The reviewed residual tests explored various ways of estimating the residual capacity of concrete members being subjected to fire. These studies investigated several NDT techniques to establish the residual concrete strength after fire exposure. A primary limitation on the reviewed experiments is that most of the specimens tested were small scale, and the results of the small-scale testing may not be representative of full scale members. Another limitation is that the fire exposure used in each test were of standard fires and do not necessarily represent a realistic (or design) fire. The limited number of available experimental studies that investigated the residual capacity of concrete members after fire exposure further demonstrates the knowledge gap that exists in understanding their behavior after fire exposure.

Table 2.1. Experimental Studies on the Residual Capacity of RC Columns after Fire.

Study	Objectives/Detail	Features and Methodology	Observations/ Conclusions	Strengths/ Draw-Backs
Lie, Rowe, Lin (1986)	• To establish a reliable means of quantifying the residual capacity of RC columns after fire exposure for practical use.	• Two RC columns made normal strength concrete were fabricated (305 x 305 x 3810 mm). • Each column was measured with a UPV instrument using the direct transmission technique (transducer placed on one face of column, and receiver placed on the opposite face). • Each column exposed to a standard ASTM E119 fire with a defined cooling phase. • The residual capacity of each column was established by direct	• The RC columns showed a remarkable difference between the UPV measurements taken before and after fire exposure. • Correlation of pulse velocity and concrete strength is dependent on many factors and will vary between concrete mixes. • UPV measurements can give insight into the quality of fire damaged concrete.	• The relative humidity of the tested columns was high (80-90%), which could be the cause of spalling that was observed during fire testing. • UPV measurements made were direct transmission, which can be heavily influenced by reinforcing steel, and may not necessarily give reliable results.

Table 2.1 (cont'd).

		loading of the columns until failure.		• Consideration was given to the cooling phase after fire exposure, which has not been extensively studied.
Colombo and Felicetti (2007)	• To identify quick and easy methods in evaluating the thermal damage that an RC structure has been subjected to. • In-situ viability of the proposed NDT techniques is	• Concrete cubes were fabricated evaluate the sensitivity of each of the proposed NDT methods to different levels of temperature. • Several concrete panels (275 x 550 x 80 mm) cast were exposed to fire on a single side of the specimen. Used to evaluate how each NDT method can detect depth of thermal	• Indirect UPV measurements proved to be very sensitive do to the effect that heating has on the dynamic modulus of concrete as well as the moisture content. UPV measurements also require a flat surface which may not always be present in a fire exposed concrete surface. • The approach used to employ colorimetry proved	• Only studied siliceous aggregates. This is particularly important with respect to the colorimetry testing, because calcareous and igneous aggregates color changes due to temperature exposure is less pronounced. • Digital camera

Table 2.1 (cont'd).

	also explored. • The NDT techniques studied were that of UPV, Digital camera Colorimetry, and drilling resistance.	damage in a specimen. • A concrete electrical duct was exposed to an ISO 834 fire. The intent was to simulate a realistic structure and evaluate the concrete properties after fire exposure.	powerful because of the well-known color changes of heated concrete. Fire exposure temperatures can be readily established from color of aggregates and cement paste of a sample.	colorimetry also requires extraction of a core from a structure to perform the testing, which may not always be possible/viable.
			• Monitoring drilling resistance in a fire damaged concrete surface yielded reasonable results in assessing the severe damage gradients occurring in a concrete structure during fire. The method proved to be fast in conducted in in-situ conditions.	

Table 2.1 (cont'd).

Kodur, Raut, Mao, Khaliq (2013)	• To evaluate the residual capacity of RC columns after fire exposure. To present a post-fire assessment approach for residual strength evaluation.	• Five full scale RC columns (203 x 203 x 3350 mm) were fabricated and exposed to a design fire and controlled cooling phase. • One column was normal strength concrete and the remaining four were high strength concrete (HSC). After cooling, the columns were loaded until failure.	• RC columns exposed to realistic fires can retain most of their original capacity, particularly if the column does not experience fire induced spalling. • HSC is more likely to spall; this can be mitigated by the addition of fibers (synthetic, steel, and hybrid).	• Only one NSC column was studied. • A theoretical method is recommended to be used in estimation of the maximum rebar and concrete temperatures (needs further validation).
			• Rebar temperatures is the governing parameter for the residual load carrying capacity of fire exposed RC columns.	

2.3 Numerical Studies

A review of literature indicates that a substantial number of analytical studies have been conducted to characterize the behavior and fire resistance of RC columns. However, there are very limited studies on the residual capacity of fire damaged RC columns. The primary objectives of the first group of studies was to find the response of RC columns during fire, and the second group was to establish their fire resistance through mathematical models. The primary findings of each study are summarized below. Table 2.2 summarizes the objectives, test parameters, test methods/features, test strengths/drawbacks (if any), and primary conclusions.

2.3.1 Studies by Cioni, Croce, and Salvatore (2001)

This study assessed damage that RC elements can experience under fire conditions. The proposed method consisted of determining the maximum fire temperature that an RC member has been exposed to. To do this, the researchers used a mineralogical analysis on the crystalline phase with diffractometry. Dolomite ($CaMg(CO_3)_2$), and calcite ($CaCO_3$) are the most temperature sensitive minerals in calcareous concrete at elevated temperatures [14]. At 832ºC dolomite dissociates into calcite, carbon dioxide (CO_2) and periclase (MgO) [14]. The reduction of relative intensity of dolomite and increase in calcite were used as a marker of an isotherm in the RC member cross section. This marker temperature was then used as the maximum fire temperature that the concrete member experienced at the depth of observed reduction of relative intensity in the sample.

The surface fire temperature was then reconstructed by using the maximum temperature at a known depth in the cross section of the member, as described above. This was done by using a finite element model to determine the time into fire exposure at which the temperature in the section at the previously determined depth was achieved. From this information, a time-temperature fire exposure curve was established and the evolution stresses and strains in the member were studied

through thermo-mechanical analysis. By analyzing the stress-strain state with the predicted fire exposure curve, the relative degree of structural damage was estimated. It should also be noted that the analysis that was performed in this study was on a two-dimensional cross section of the structural member studied.

2.3.2 Studies by Zha (2003)

Zha (2003) [54] focused on studying the behaviors of RC members under fire conditions by employing a finite element model discretized with three dimensional non-linear finite elements. The temperature distribution in the section of concrete members is calculated by the Hertz's simplified method (established in 1981), which is then used as an input to the finite element program developed [54]. The Hertz's simplified approach calculates the temperature distribution in a material as a function of depth and time of fire exposure. This function is the summation of three separate functions in which the dependent variables of each are depth in the material cross section and time of fire exposure. The three functions are the solutions to the Fourier Heat Transfer Equation [54].

A time dependent thermal stress analysis was then performed using the above noted temperature distribution as an input for the model. Analyses were conducted on both rectangular RC column and beam sections. The developed model exhibited a good prediction of fire resistance and performance of concrete members under fire conditions. This claim was validated by showing unremarkable differences between the developed model and standard empirical approaches (British Standard 8110: Part 2 and Eurocode 2, [20]) which determines the fire resistance of concrete members.

2.3.3 Studies by Bratina, Cas, Saje, and Planinc (2005)

Bratina, et al. (2005) [9], developed a two-step finite element model for the thermo-mechanical non-linear analysis of the behavior of RC columns during fire. The first step in analysis was to determine the distribution of temperature from certain durations of fire exposure. The next step incorporated mechanical analysis in which the temperature distributions are applied as loads from the previous step. The authors utilized what they term as a, "strain-based planar geometrically exact and materially non-linear beam finite elements" to model the columns [9]. These elements are used to satisfy the conditions that the equilibrium and axial forces and bending moments coincide at the integration points.

The primary findings from comparison of the numerical results from this model to the predictions of the European Building Code, Eurocode 2 (2002) [20] were in good agreement in the prediction of fire resistance. It should be noted, however, that there was a significant difference between the predicted and experimental axial deformations. However, when compared to experimental results, there was a significant variation between the predicted axial deformation and the experimentally measured displacements. The authors attributed this difference to the thermal, creep and transient strain components not being sufficiently accurate in describing the materials employed in experimental studies they conducted [9].

2.3.4 Studies by Gernay and Dimia (2011)

Gernay and Dimia (2011) [23] developed a numerical model to predict the structural behavior of RC columns subjected to a natural (or realistic) fire, and attention was specifically given to the decay phase of fire exposure. A two-dimensional finite element analysis was performed on various cross sections. The main objective of the study was to get an insight to the parameters and conditions that could lead delay the collapse of a column after fire exposure. The parameters

considered are the duration of the heating phase of the fire, effective length of the column, and the section size of the column.

The main findings of the numerical study are that a failure during the cooling phase of a fire is possible in RC columns. Temperatures in the central portion of the modeled RC columns continue to increase even after the heating phase of a fire scenario has ceased. A failure during the cooling phase can also be ascribed to the additional strength loss of concrete that will occur during the cooling phase, when peak concrete temperatures are reached. The most critical situation for a delayed failure are a short fire exposure and for columns that have a low slenderness ratio (short length and/or massive cross sections) [23].

2.3.5 Summary

From the presented review of literature, it becomes clear that much of the focus of the numerical studies carried out concentrates on the fire resistance and behavior of concrete members during fire. A limited number of methods predict how a RC member, specifically RC columns, will behave after exposure to fire. Much of the methods were carried out on planar sections and neglect the full-size member behavior. This limitation does not allow for the prediction of axial deformations in a full-sized RC member under a fire scenario. Further, many of the studies were under standard fires and do not incorporate realistic design fires.

Table 2.2. Analytical Studies on Post Fire Capacity of RC Columns.

Study	Objectives/Detail	Analysis/Model Features	Observations/ Conclusions	Strengths/ Draw-Backs
Cioni, Croce, and Salvatore (2001)	Combined experimental data and a numerical model to establish the thermo-mechanical response of RC structures and the relative degree of damage from fire exposure.	• Employs a finite element method • Obtain the thermal profile on the RC structure from a mineralogical analysis. • Using the thermal profile, a fire exposure curve is generated and the stress-strain evolution of the structure is obtained.	• Two-stage cracking can be discerned from the stress fields obtained during fire exposure. • The cracking stages give insight to how severe a crack is based on the stress level in the model.	• Requires a physical sample to perform the mineralogical analysis; can be destructive to the structure and time consuming. • Performed only a small sectional analysis using solid elements.

Table 2.2 (cont'd).

Zha (2003)	Focuses on an analytical method to establish RC column behavior during fire exposure.	• Uses a three-dimensional finite element method to determine the fire resistance of an RC structure. • Uses the Hertz Method (1981) for calculation of the temperature distribution in the RC member during fire exposure.	• The model showed a good prediction of fire resistance of RC structures. • This was corroborated by comparing the analysis results to that of standard approaches available in practice, which showed unremarkable differences in fire resistance.	• Analysis only conducted considering thermal stresses with no mechanical load applied. • Model needs to be validated with experimental studies. • Only conducted analysis during fire exposure with no consideration given to the decay phase.

Table 2.2 (cont'd).

Bratina, Cas, Saje, Planinc (2005)	Develop a two-step analytical method to determine the fire resistance of fire exposed RC columns.	• Utilizes planar beam elements in a finite element method. • First stage of analysis determines the temperature distribution from various fire exposures. • The second stage is a mechanical analysis performed by using the temperature distribution applied as loads from the previous step.	• The predicted fire resistance of the analyzed members was agreeable to the currently available calculations methods in codes and standards. • The predicted axial displacements from the analysis did not agree well to experimental studies conducted. Authors indicate that further refinement of the developed method is needed.	• Consideration was again only given during the fire exposure phase, and not the post fire behavior. • Thermal response of RC structures is agreeable, but the mechanical response is not agreeable to experimental data.

Table 2.2 (cont'd).

Gernay and Dimia (2011)	Establish a numerical model that predicts the structural behavior of RC columns during realistic fires and after fire exposure.	• Uses a two-dimensional finite element method. • Analysis conducted on several different sizes of cross sections. • Parameters varied were duration of heating phase of fire, effective length of column and the section size.	• Analysis indicated that failure during the cooling phase of a fire is a possible event. • This is attributed to temperatures in the central portion of the concrete column can keep increasing even after the external column temperatures have fallen back to ambient conditions. • The most critical situation for a delayed failure is a short fire exposure with high temperatures (like a hydrocarbon fire) on a column with a low slenderness ratio.	• Does not consider a three-dimensional geometry in the analysis. • Authors indicate that further validation is needed against experimental studies.

2.4 Codes of Practice

There is limited guidance in the assessment of fire damage in RC structures. The review below focuses on the methodologies and recommendations that are available in codes and standards on residual capacity of RC members. The reviewed documents are ACI 562-16 [3], and a fire protection planning report prepared by the Portland Cement Association (PCA) [47].

2.4.1 ACI 562-16: Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures and Commentary

This provisional standard from the American Concrete Institute (ACI) furnishes requirements and recommendations for a multitude of types of damage that a concrete structure can experience in its lifetime. This standard specifically establishes assessment and repair requirements for fire exposed concrete members. The assessment recommends an investigation and review of the damaged structure, its plans, construction data, reports, local jurisdictional codes, and other available documents of the existing structure. Specific requirements are defined for fire damaged structures.

ACI 562-16 [3] stipulates that the assessment of fire damage or other deterioration mechanisms that results in a change of the material properties (such as compressive strength and modulus of elasticity) are required [3]. Evaluation of the extent of fire damage to the material properties of structural members and how that damage will influence the performance of the structure is essential. ACI 562-16 [3] permits use of NDT methods to obtain residual material properties of fire damaged members.

Nonetheless, ACI 562-16 [3] explicitly states that use of NDT methods alone are not adequate to characterize concrete strength. Core samples are required from a damaged RC member to establish its compressive strength. NDT methods can be used to further quantify the material properties of

concrete after a correlation has been established between the NDT techniques proposed and the core samples obtained.

A drawback in ACI 561-16 [3] is that destructive means are required to evaluate the residual properties of concrete and reinforcing steel of a fire damaged structural member. While this is a conservative requirement, it may cause additional damage to the structure as well as it not always being possible to obtain physical samples to be tested. Further, while the standard allows use of NDT methods to compliment physical samples obtained from the fire damaged member, it does not recommend which NDT methods to use.

2.4.2 Portland Cement Association Fire Protection Planning Report (1994)

An approach is detailed in this report to assess the extent of damage to fire exposed structural members made of concrete. Determining whether reinforcing steel has been exposed directly to fire because of spalling is an important factor in determining the extent of damaged to an RC member. The exposed reinforcing steel in an RC member from spalling is an indicator that the reinforcement has been exposed directly to fire temperatures. This is a major concern because reinforcing steel can lose up to half of its room temperature yield strength at temperatures more than 600ºC [47].

In the case of RC columns, particularly in those that have a high density of lateral ties or spiral reinforcement, the possibility of reinforcing steel reaching 600ºC is great. This remains true even in the absence of severe distortion or buckling. RC columns that are suspected to have been exposed to fire warrants a more detailed investigation of said member. The report indicates use of both NDT and destructive means to establish the extent of damage that an RC member has experienced.

Three NDT methods are discussed and they include use of Ultrasonic Pulse Velocity (UPV), Impact Echo (IE), and Impulse Radar (IR) technologies. The UPV method measures the velocity of sound through a concrete cross section. The sound velocity can give insight into the concrete quality as well as an estimation of concrete strength. The IE technique involves use of an impact hammer to send a low frequency stress wave into the concrete. The wave energy is then reflected to a sensor. The data can be used to detect, locate and classify discontinuities such as voids, delamination's, cracks, and bond loss between cement paste and aggregates within the member. In the IR technique, both magnetic and microwaves are propagated into the concrete section. This can be used to locate reinforcing steel and other embedment's in the concrete, and establish the thickness of structural components as well as detect the presence of voids. This can be particularly useful in determining the thickness of undamaged concrete over steel reinforcement in cases where fire damage has not been fully extended into the steel.

Recommended destructive test methods include extraction of concrete cores and reinforcing steel from damaged concrete members. The extracted specimens can be characterized through laboratory testing. The destructive methods will provide the most accurate assessment of the concrete and steel strength of the damaged member. When the residual steel and concrete strength of a fire damaged structural member has been determined, an appropriate rehabilitation technique can be selected. Alternatively, if the damage to the structure is determined to be severe enough, partial or complete replacement of the structure may be warranted.

2.4.3 Summary

There is little guidance available to practitioners in assessing the extent of fire damage to RC structural members. The standards available present several ways to determine residual concrete strength. The primary recommendations (namely ACI 562-16 [3]) are to use destructive or semi-

destructive testing to determine the residual strength of fire damaged concrete and reinforcing steel. This type of testing is not always viable, and has the potential to further damage an RC structure. The sole use of NDT techniques is prohibited. This means that there is a gap in knowledge with respect to the reliability of the available NDT methods, and further research with the use of these techniques is warranted.

2.5 Factors that Influence Behavior of RC Columns in Fire

Upon examination of the results of experimental and numerical studies, it becomes obvious that conventional concrete generally exhibits good fire performance. However, due to the demonstrated lack of studies that have been conducted on the residual capacity of RC columns after fire exposure, the residual behavior of RC columns is not well understood. Key factors that influence the fire performance of RC columns are discussed in the following sections.

2.5.1 Concrete Moisture Content

The moisture content in concrete is generally expressed as relative humidity (RH). The RH level in concrete can have an influence on spalling. Higher values of RH can lead to a higher incidence of spalling [1].

2.5.2 Concrete Strength

RC columns made of NSC can exhibit good fire resistance up to three hours (or greater, in some cases), even under full service loads [48]. However, HSC (generally above 70 MPa) could exhibit a lower fire resistance because the HSC is more susceptible to explosive spalling during a fire [38]. Due to the hydration reaction of cement and water never ceasing, older concrete may exhibit compressive strengths at the levels of HSC. This means that older NSC could exhibit explosive spalling in a fire.

2.5.3 Concrete Density

Concrete density can have an influence on the behavior of RC members subjected to fire. The incidence of spalling in lightweight concrete has shown to be much higher than that of NSC [30]. This is attributed to a higher amount of free moisture present in lightweight aggregates, which creates a higher vapor pressure under severe fire exposures, which in turn leads to fire-induced spalling.

2.5.4 Fire Exposure Intensity

A high heating rate of a fire, which is common in hydrocarbon fires, can lead to a lower fire resistance of concrete. The intensity of a fire relies on several factors, but the primary variables that will influence fire intensity and duration are fuel available, ventilation of the structure and active fire prevention systems present (e.g. fire brigade intervention and/or sprinklers). A fire that heats very quickly, and allowed to burn for a long period, will have negative effects on RC structures.

2.5.5 Column Dimensions

RC columns made of NSC that are more massive will have a higher fire resistance than that of a smaller RC column. This implies that larger columns exposed to more severe fires could potentially show a higher residual capacity than that of a smaller RC column under the same fire conditions. This is because the thermal damage to the RC column is less pronounced in large sections, due to the insulative properties of concrete.

2.5.6 Lateral Ties

The results from several studies show that the configuration of lateral reinforcement and confinement of concrete will have a significant influence on the fire resistance of RC columns. Greater fire resistance is achieved in RC columns by use of lateral ties that are bent at 135^{o} into

the core of the column, as well as closer spacing of the lateral ties themselves. The extent of spalling in columns with bent tie configuration is relatively less compared to columns without a bent tie configuration. Columns with 135º bent ties will also exhibit a pyramid compression failure pattern with the failed section being confined within one or two tie spacings [4].

2.5.7 Load Intensity and Type

Research has shown that the magnitude of load and how that load is applied will influence the incidence of spalling in concrete and consequently its resistance to fire [37]. The presence of high levels of load on RC columns will cause it to be more susceptible to spalling because the member is subjected to mechanical stresses due to axial load in addition to the internal stress build up in the form of pore pressure increase from steam. The incidence of spalling will be higher in columns that are eccentrically loaded since this type of loading will induce additional tensile stresses in the section of the member [38].

2.5.8 Aggregate Type

There are two primary aggregate types used in concrete; carbonate aggregates predominately made of limestone and siliceous aggregates predominately made of quartz. Generally, fire resistance of RC columns made with carbonate aggregate concrete is approximately 10% higher than RC columns made with siliceous aggregates [33]. The use of each aggregate type will depend on several factors, such as availability and geographic location of the construction site of a RC structure.

2.6 Knowledge Gaps

The reviewed literature demonstrates that there have been a limited number of studies conducted on evaluating the behavior of RC columns after fire exposure. To predict the residual capacity of a fire damaged RC column, a more detailed analysis procedure is required that includes fire

exposure on different sides of a column, spalling, creep and other features that includes realistic fires and as well as different strain components (mechanical, thermal, transient, and creep). The future work in this area needs to be focused on developing detailed analytical models which should be capable of accounting for these factors. Additional exploration of numerical studies will facilitate more reliable standards in determination of a RC columns post fire residual capacity. The following are some of the tasks required to close the current knowledge gaps on determining the residual post fire capacity of RC columns:

1. There is a lack of test data on the thermal and mechanical response of RC columns subjected to a full heating and cooling phase. Data from realistic fire exposure experiments is critical in the development of rational approaches to determine the residual capacity of RC columns. Additionally, this data will help validate any developed numerical models that predict the residual response of a fire exposed RC column.
2. The present analytical and numerical approaches to evaluate the residual capacity of a fire exposed RC member are reliant on the peak temperatures experienced in the member alone. The distinct material properties exhibited by concrete during the cooling and residual (after cooling) phases of fire exposure are not accounted for.
3. The strain hardening in reinforcing steel as well as tension stiffening exhibited by concrete is ignored in most approaches in estimating the residual response of fire damaged RC members. Inclusion of these parameters may improve the accuracy of predicted residual behavior of RC columns.
4. A general approach or guidance in developing a reliable numerical model for tracing the residual response of fire exposed RC columns does not exist.

5. The influence of critical parameters such as fire exposure scenario, load level, restraint level and cross-sectional size of fire exposed RC columns has not been adequately quantified in previous studies.

CHAPTER 3

3 EXPERIMENTAL STUDY

3.1 General

The state-of-the-art review (Chapter 2) revealed that many fire tests have been carried out to evaluate behavior of RC columns during fire. However, very few experimental studies have been conducted to evaluate the post-fire response of RC columns. Some of the drawbacks of the studies conducted is that they do not represent realistic stress states or failure modes as encountered in RC columns exposed to fire.

At the structural level, it was observed that most tests on RC columns adopted a standard heating phase and without any load acting on the columns during fire exposure, which is not representative of a realistic (design) fire scenarios and stress conditions. Additionally, the temperature and deflection history during fire exposure, especially during the cooling phase which is critical for establishing the post-fire response of the column, was not reported in the majority of the studies reviewed. Further, local response and crack patterns were not reported in any of the residual capacity tests carried out.

Thus, to generate the needed data to further validate and refine the numerical model, full scale fire resistance tests followed by residual capacity tests were undertaken as part of this Thesis. Details of fabrication, fire test (elevated temperature exposure) procedure, residual tests procedure, measured response parameters, and results from the tested RC columns are presented below.

3.2 Experimental Details

The experimental program consisted of realistic fire exposure tests on two reinforced concrete columns, made of NSC. The tested columns have the same geometry, reinforcement, and mix design. One of the two columns were approximately eight years old at the time of testing, and was stored outdoors at the Civil Infrastructure Laboratory at MSU. The following sections detail the features of the experimental program.

3.2.1 Test Specimens

The experimental program at the structural level consisted of conducting residual capacity tests on two RC columns, herein designated as C-1 and C-2, after exposing them to design fire scenarios under realistic load levels. Column C-1, which is the older column of the two, and column C-2, which was cast in January 2017, were each fabricated with normal strength concrete (NSC). Each column is 3350 mm long and of a square cross section of 203 mm. These columns were designed as per ACI 318 specifications [2], and the details of the column design can be found in Appendix II.

Individual columns had four 20 mm diameter (#6) bars as longitudinal reinforcement. Each had 10 mm diameter (#3) stirrups, with a spacing of 200 mm center to center along the length of the column. The stirrups were bent at 135^{o} into the concrete core. The steel of the main longitudinal reinforcement and stirrups had a specified yield strength of 420 MPa. Based on the tensile strength tests carried out on the reinforcing steel used in fabrication of the columns, the yield strength, ultimate tensile strength, and ultimate tensile strain of the steel for column C-1 was 450 MPa, 705 MPa and 0.17, respectively [48]. The yield strength, ultimate tensile strength, and ultimate tensile strain for the steel in column C-2 was found to be about 431 MPa, 696 MPa, and 0.14, respectively.

Figure 3.1 shows tied reinforcing steel placed into formwork of the cast column C-2. Figure 3.2 and 3.3 depicts the elevation and cross-sectional details of the columns C-1 and C-2, respectively.

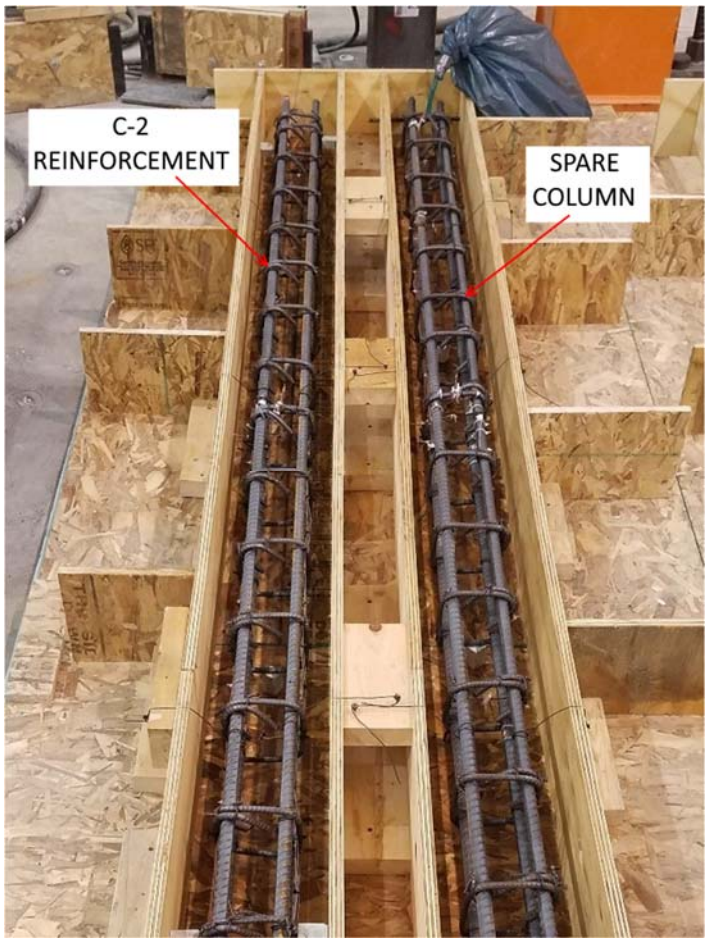

Figure 3.1. Reinforcement of Cast Columns.

One batch of concrete was used for fabricating column C-2. The older concrete column (C-1) was cast from a separate batch, but the mix design was the same as that of column C-2. The concrete batches were made with Type I Portland cement, and carbonate aggregates. Mix proportions, per cubic meter of concrete, are presented in Table 3.1. Average compressive cylinder strength of the concrete that was used to cast C-2 was measured at 7, 28 and the day of fire test of the columns is presented in Table 3.2.

Table 3.1. NSC Mix Design for Fabricated Columns.

Design Mix Proportions		
Constituent	Batch 1 [48] (Column C-1)	Batch 2 (Column C-2)
Cement (kg/m3)	390	335
Natural Sand (kg/m3)	830	790
Limestone (kg/m^3)	1037	1032
Water (kg/m3)	156	151
Max W/C Ratio	0.45	0.45
Measured Slump (mm)	100.0	127.0
f'$_c$ (MPa)	27.6	27.6
Measured 28 Day Strength (MPa)	39	46

Each column was cast horizontally, and moist cured in the forms for 7 days (refer to Figure 3.1 for an example of the formwork used to fabricate each column). Each specimen was lifted from the formwork and stored in air maintained at about 25ºC and 40% relative humidity. The average compressive cylinder strength of the concrete, measured at 28 days, was 39 MPa and 46 MPa, for C-1 and C-2, respectively. The test day compressive cylinder strength was measured to be 49 MPa and 62.1 MPa, for C-1 and C-2, respectively. The higher value of compressive strength observed for column C-2 is attributed to excess cement being batched by the ready-mix producer for Batch 2. The moisture condition (relative humidity) was measured at a depth of 50 mm of each column surface using a relative humidity probe on the day of each fire test. The relative humidity of each column on the day of fire testing can be seen in Table 3.1.

3.2.2 Instrumentation

Instrumentation for the columns included thermocouples, strain gauges and displacement transducers. Type-K Chromel-alumel thermocouples, 0.91 mm diameter, were installed at the mid height of column C-1. Column C-2 had thermocouples installed at the mid height, quarter height and three-quarter height cross section. The thermocouples were set to measure concrete temperatures at various depths as well as rebar temperatures. C-1 consisted of seven thermocouples, while C-2 consisted of twelve.

Strain gauges were also mounted on the reinforcing steel at the mid height of each column. The location and number of thermocouples and strain gauges in the cross section for column C-1 is shown in Figure 3.2. Figure 3.3 shows the thermocouples and strain gauge locations in the cross section of column C-2. Figure 3.4 depicts an example of the installation of the thermocouples and strain gauges in column C-2. Axial deformation of each column was measured at the top of the column using linear variable differential transducers (LVDT's) with gauge length 375 mm. Axial deformations were recorded both during fire as well as during the residual capacity testing using the same LVDT's.

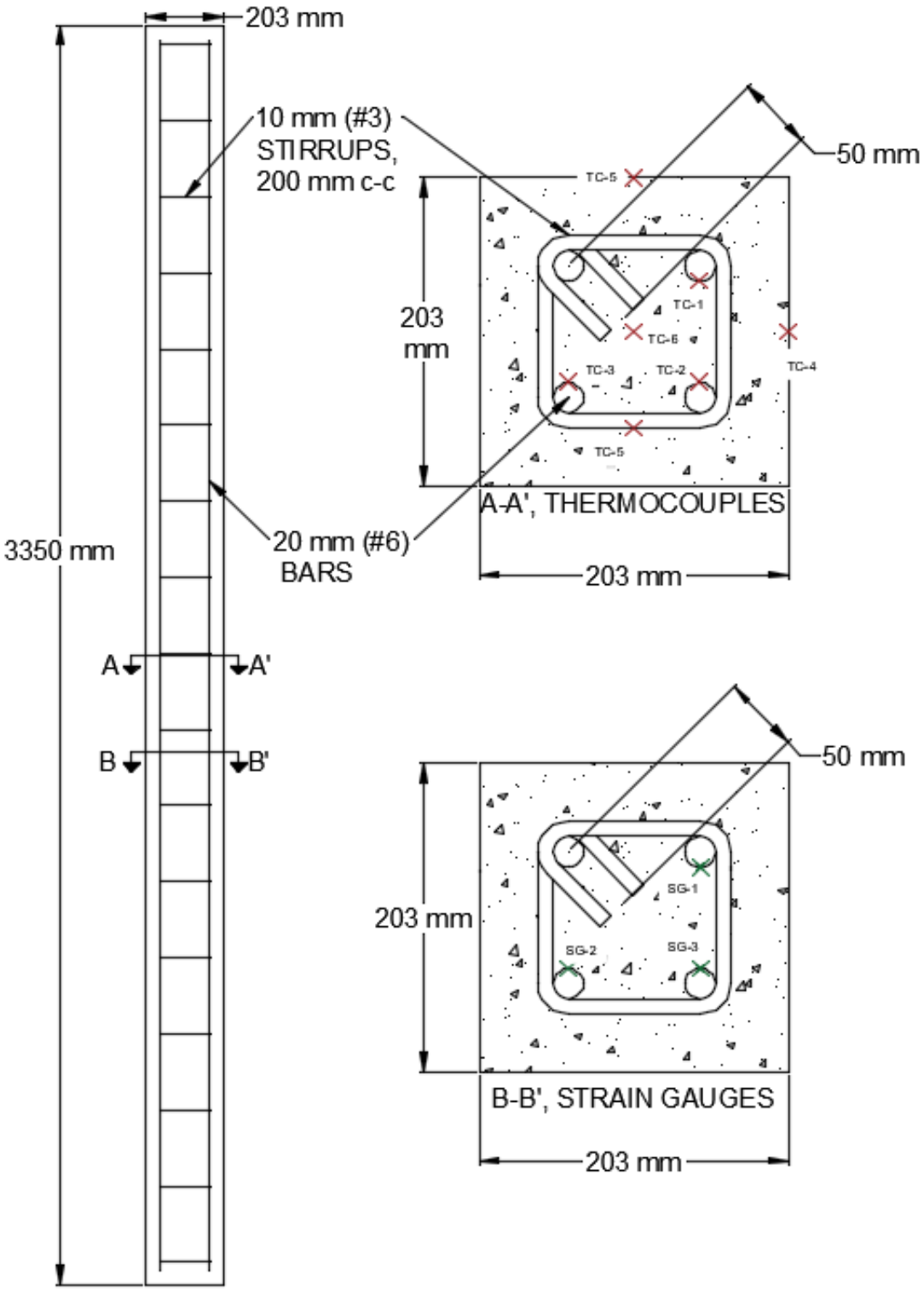

Figure 3.2. Column C-1 Design and Instrumentation [48]; Not to Scale.

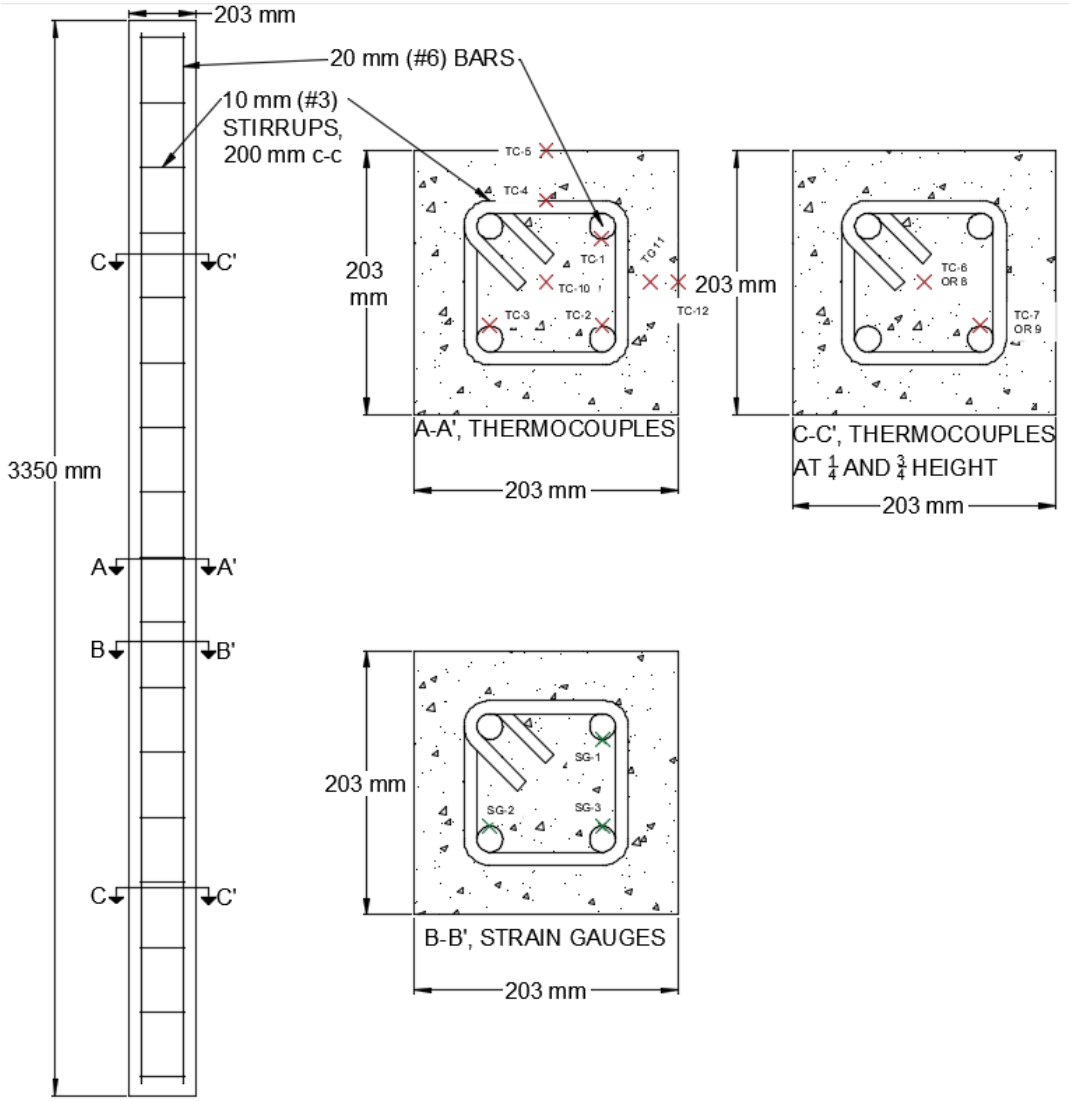

Figure 3.3. Column C-2 Design and Instrumentation; Not to Scale.

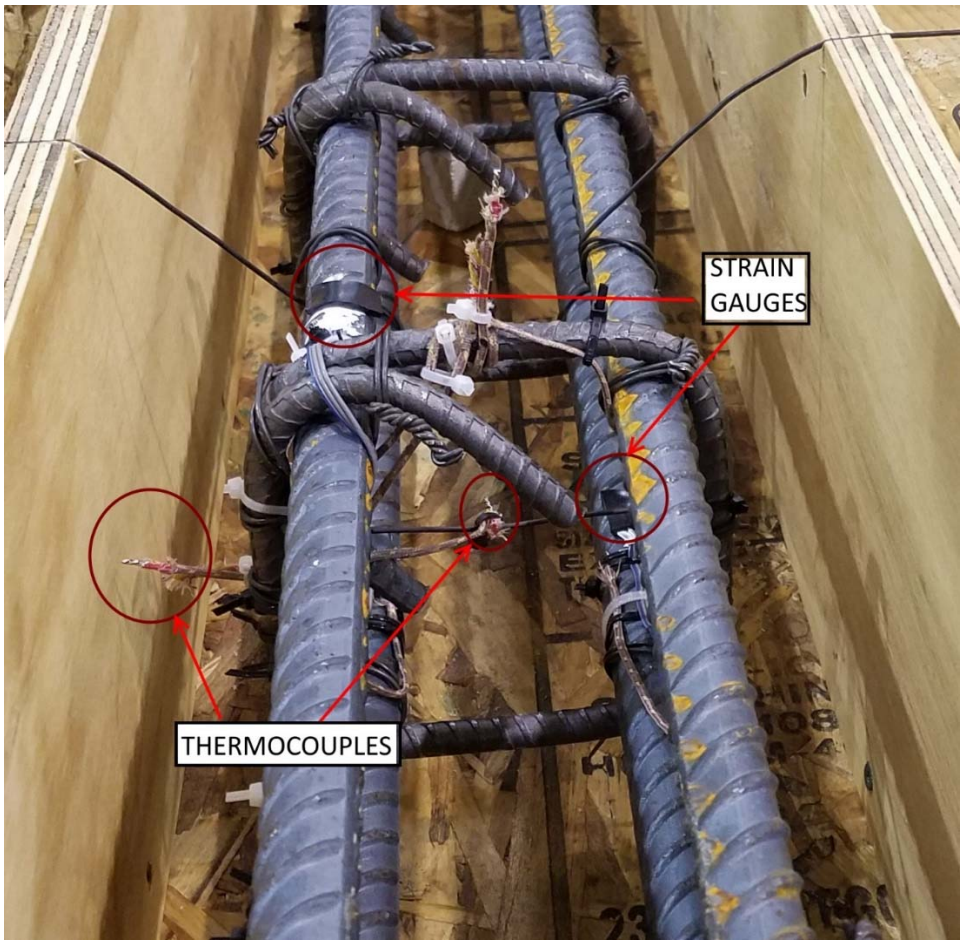

Figure 3.4. Strain Gauge and Thermocouple Installation (Column C-2).

3.2.3 Fire Resistance Tests

Fire resistance tests on the RC columns were carried out using a structural fire testing furnace located in the Civil Infrastructure Lab at Michigan State University. This furnace has been specially designed to produce conditions, such as temperatures, loads and heat transfer, to which a structural member may be subjected to during a fire. The furnace, shown in Figure 3.5, has the capacity to supply both heat and applied loads that may be present in a typical building exposed to a fire.

Figure 3.5. Structural Fire Testing Furnace.

3.2.3.1. Test Furnace:

The furnace consists of a steel frame supported by four steel columns, with a fire chamber that is 2.44 m wide, 3.05 m long and 1.68 m tall. The maximum heating power the furnace can supply is 2.5 MW. Six natural gas burners located within the furnace provide thermal energy, while six type-K Chormel-alumel thermocouples, as per ASTM E119, distributed throughout the test chamber, monitor the furnace temperature during a fire test. The furnace temperatures obtained during a fire test are used to manually adjust fuel supply, and maintain a temperature course consistent with either a standard or realistic design fire. Two small viewports on opposite sides of the furnace walls provide a way to visually monitor fire-exposed specimens during testing. The furnace can accommodate two columns at a time and different load levels can be applied on each column.

Fire testing was carried out by placing one RC column in the furnace and exposing the column to a desired fire. The central 1.7 meters of the column height was exposed to fire, as seen in Figure 3.6.

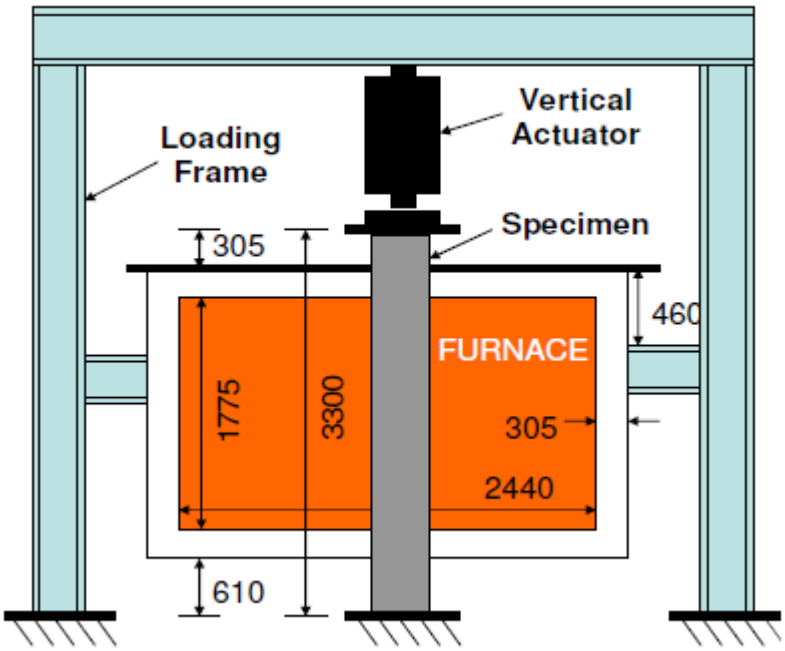

Figure 3.6. Structural Fire Testing Furnace Dimensions (in mm).

3.2.3.2. Test Procedure:

To investigate the effect that different fire scenarios has on RC columns, columns C-1 and C-2 were tested under two different design fires. Column C-1 was exposed to a design fire (DF-1) that featured a 90-minute heating phase that followed that of ASTM E119. At completion of the heating phase, the furnace temperature was controlled to follow a decay phase of 4.5°C per minute until room temperature at the column surface was achieved. A load level of 50% of the factored ACI design capacity was maintained during heating and cooling phases. Column C-2 was exposed to a similar design fire (DF-2) which had a 120-minute heating phase with the same cooling regimen as in DF-1. A load level of 55% of the factored ACI design capacity was maintained in each phase

of DF-2. The load controlled method was used in application of load for each case. Each design fire can be seen in Figure 3.7, in which fire (furnace) temperature is plotted as a function of time.

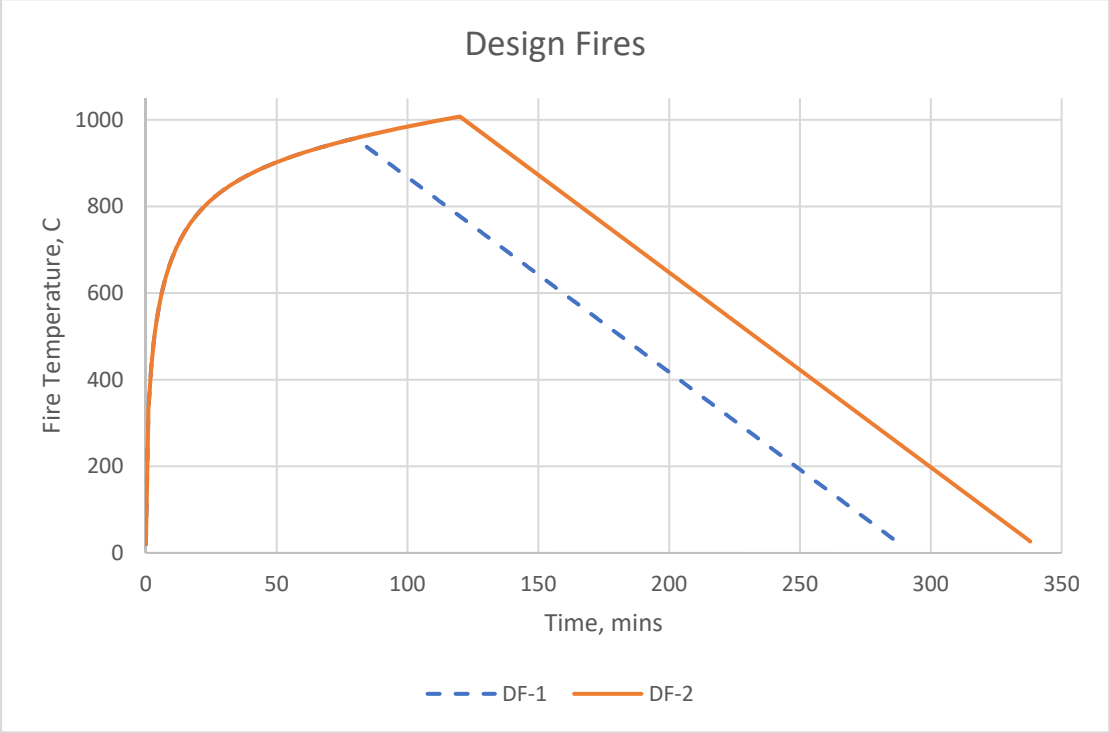

Figure 3.7. Design Fires for Fire Testing.

Each column was slender as per ACI 318 and were tested under a concentric axial load. The tolerance of eccentricities at the top and bottom of the columns tested were ±5 mm. The type of fire exposure and load ratio for each column are tabulated in Table 3.2. The load ratio is the ratio of the applied load during fire testing to the column capacity as computed per ACI 318.

A concentric axial load was applied at the top of each column approximately 30 minutes before the start of each fire test and was maintained until a condition was reached at which no further increase of the axial deformation of the column could be measured. During the fire test, the column was exposed to heat controlled in a way that the average temperature of the furnace was followed, as closely as possible, to the targeted fire scenario. The aforementioned load levels were

maintained constant throughout each fire test duration, including the cooling phase. The column was considered to have failed if the loading jack could no longer maintain the desired applied load.

3.2.4 Measured Response Parameters

During the fire test, each column was exposed to heat controlled in such a way that the average furnace temperature followed, as closely as possible, the target time-temperature curve for the test. The load was maintained constant through the duration of fire testing. The test data collected was the furnace temperatures, specimen temperatures, strains, axial deformations, and loading applied. Test data was recorded every five seconds through the data acquisition system. Observations were made every five minutes through the view ports in the furnace to record any major changes in the specimens such as fire induced spalling or visible cracks. The column was considered to have failed if it could no longer sustain the applied load from the jack.

After the fire testing, a post-fire inspection was carried out to record the presence of any fire induced spalling, and concrete cracking in the fire exposed columns.

3.2.5 Residual Capacity Test

After each column reached ambient temperatures, a residual capacity test procedure was employed approximately 48 hours after the completion of the heating phase for each column. Prior to residual capacity testing, non-destructive evaluation of each fire exposed column was conducted. The method employed in non-destructive evaluation are both visual inspection and indirect ultrasonic pulse velocity (UPV) measurements. The residual testing procedures are presented below.

3.2.5.1 Non-Destructive Testing

UPV testing has shown promising results in the characterization of fire damaged concrete as it has been the subject of several research programs [21,8,15,40]. One objective of each study has been to investigate the sensitivity of compressive strength degradation with respect to pulse velocity

degradation in fire damaged concrete members. Figure 3.8 summarizes the upper and lower bounds of the trends in UPV degradation reported by other researchers [21]. The average value plotted between the upper and lower bounds is used to estimate the relative loss in concrete strength based on pulse velocity degradation. The decay in UPV values between the damaged and undamaged concrete is first computed. Then, using this degradation ratio, the percent reduction of concrete compressive strength is estimated using the plot of average values presented in Figure 3.8.

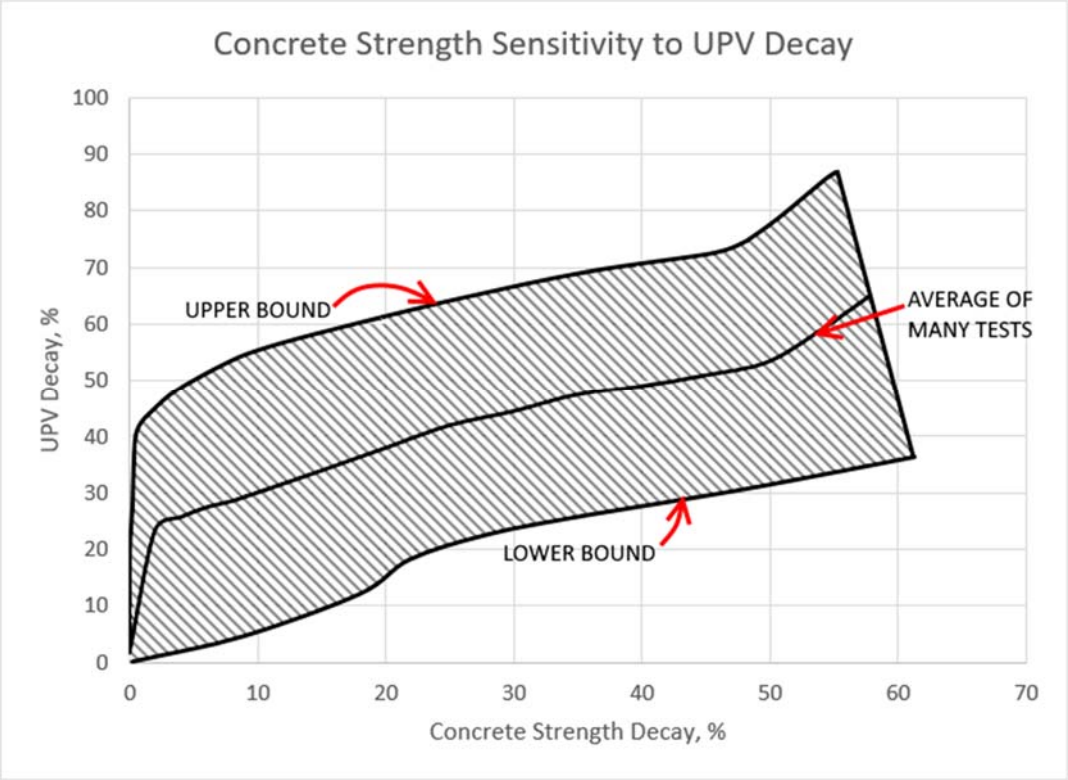

Figure 3.8. Concrete Compressive Strength Sensitivity to UPV Decay, adapted from Felicetti [20].

The pulse velocity of concrete in both damaged and undamaged states can be provided via the indirect UPV technique, which is based on the refraction of longitudinal ultrasonic waves [15]. Indirect measurements of pulse arrival time are performed by holding a transducer that emits sound waves at a known frequency on the face of a concrete element. A receiving transducer is held on

the same face of the concrete element (see Figure 3.9). The pulse travel time will rise with increasing depth, which is the rule after fire exposure to a concrete member. It is implied, therefore, that lower pulse velocity values observed in the layers of concrete closer to the surface correspond to concrete that has experienced higher temperature exposure. Deeper concrete, that has not been as adversely affected by elevated temperatures, will have higher pulse velocity values. The measurement of pulse arrival times with increasing distance along the surface of a concrete specimen allows deeper and deeper material layers to be investigated. A schematic of pulse velocity measurements of a fire damaged concrete surface is presented in Figure 3.10, derived from Colombo & Felicetti [15].

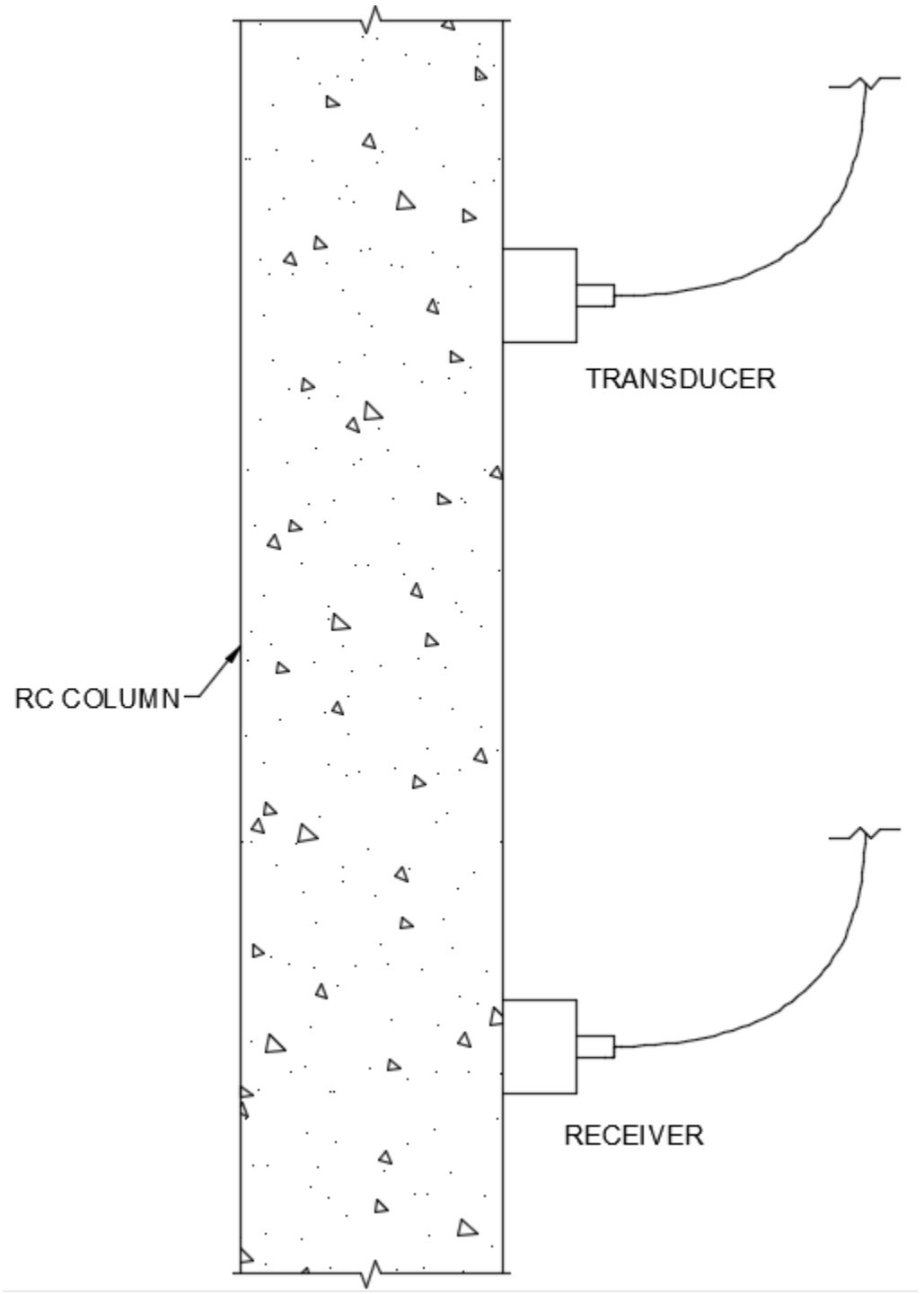

Figure 3.9. Indirect UPV Measurement Set Up.

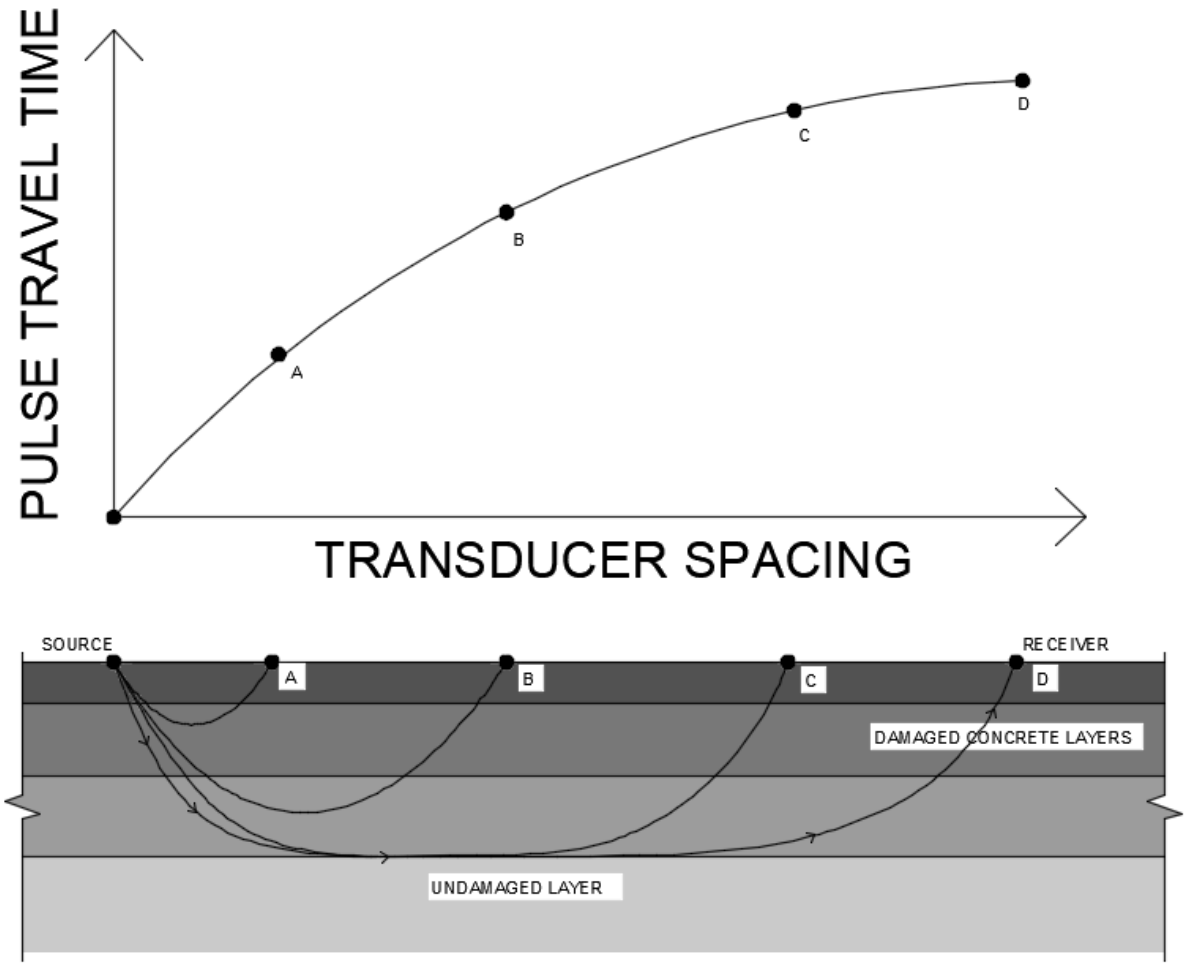

Figure 3.10. Idealized Pulse Time-Distance Plot of Fire Damaged Concrete (derived from Colombo & Felicetti [15]).

Two sets of UPV measurements were obtained for columns C-1 and C-2. The first set of UPV measurements obtained was on the undamaged surface of columns C-1 and C-2, prior to placing the columns in the furnace for fire testing. The second set of measurements obtained was after each column cooled to room temperature, and prior to residual testing. Indirect UPV measurements were obtained on the same surface as the first set of obtained UPV values. These two sets of UPV measurements are used to provide a before and after comparison of the fire exposed specimens.

3.2.5.2 Residual Capacity Test

After completion of the second set of UPV measurements, each column was then subjected to a residual load test. The column was kept under the same restraint conditions as during the applied fire. The column was loaded at an incremental rate of approximately 2.5 kN per minute, until failure occurred. Failure is said to occur when the column can no long sustain the applied load (when either buckling or crushing of the column occurs). Residual capacity testing was carried out 40 hours after completion of the fire testing (when the furnace temperature reached 25^{o}C). The residual capacity of the column is the highest recorded load that was applied just prior to failure.

3.3 Results

Data generated from the fire tests and residual capacity tests was utilized to evaluate the fire behavior and then residual behavior of RC columns C-1 and C-2. The thermal and structural response of columns is evaluated during the heating phase of fire exposure, as well as during cooling phase of fire when the column is cooling down to ambient conditions. Subsequently, residual response of the fire exposed columns is evaluated by recording post-fire residual deformations, temperature induced spalling (if present), residual capacity, load-strain response, and crack patterns as well as failure modes.

3.3.1 Behavior During Heating Phase

Each RC column was subjected to a combination of structural and fire loading during fire resistance testing. Each column was exposed to a distinct fire scenario characterized by a rapid increase in fire temperatures during the heating phase, followed by a distinct cooling phase, shown in Figure 3.7. The heating phase for DF-1 and DF-2 followed the same rate of temperature increase. The DF-2 heating period was 30 minutes longer than that of DF-1. An identical cooling rate was adopted for both fire exposures. The rate of heating and peak temperatures was different in both cases. This in turn affected both the peak temperatures experienced within the concrete and rebar,

thereby influencing thermal and structural response, as well as the residual capacity of each specimen.

3.3.1.1. Thermal Response

The thermal response of columns C-1 and C-2 is presented in Figure 3.11 and Figure 3.12, respectively, by plotting the rebar and concrete temperatures at the various thermocouple locations. The locations of the thermocouples are shown in Figure 3.2 and Figure 3.3. Unlike the fire temperatures that rose rapidly in the first few minutes of each fire exposure, the temperatures within each column remained constant. For example, temperatures within the column C-1 remain constant even as fire temperatures increased to nearly 700ºC in the first 20 minutes for each of DF-1 and DF-2. Further, a temperature plateau is seen at about 200 minutes in the center of column C-1 and about 220 minutes in the center of column C-2. This temperature plateau is a consequence of the latent heat consumed by free capillary water present in the column, as it changes state from liquid to vapor. As a majority of this pore water evaporates, the temperatures in the rebar and concrete increase with fire temperature. Furthermore, measured data indicates that temperatures in concrete, as expected, gets lower towards the inner zones of the concrete core. This can be attributed to the low thermal conductivity and high thermal capacity of concrete which slows the penetration of thermal energy to the inner concrete layers.

The effect of fire scenario on the thermal response of columns C-1 and C-2 is evident in the comparison of rebar and concrete temperatures shown in Figure 3.11 and 3.12. The fire temperatures rise rapidly to almost 700ºC in about 20 minutes of heating for both DF-1 and DF-2 fire scenarios, followed by a gradual increase to about 980ºC for DF-1 and 1000ºC for DF-2, within an additional 70 minutes and 100 minutes of fire exposure, respectively, followed by decay (see Figure 3.19 and Figure 3.20). Correspondingly, for each column, the recorded temperatures in

concrete and steel rebar increased to a maximum value and then decreased. However, the temperature within the columns continues to rise even during the decay phase of the fire exposure. In fact, rebar temperatures begin to decay approximately 90 minutes after completion of the heating phase of DF-1 and 80 minutes after completion of the heating phase of DF-2 (see Figure 3.19 and Figure 3.20). Nevertheless, cross sectional temperatures in the columns begin to decrease after attaining a peak value due to the presence of a decay (cooling) phase in the time-temperature exposure of the design fire, which leads to cooling in the RC column. The peak rebar and concrete temperatures recorded in column C-2 were higher by nearly 100ºC, when compared with the measured temperatures within column C-1. This can be attributed to the longer heating duration of the design fire DF-2.

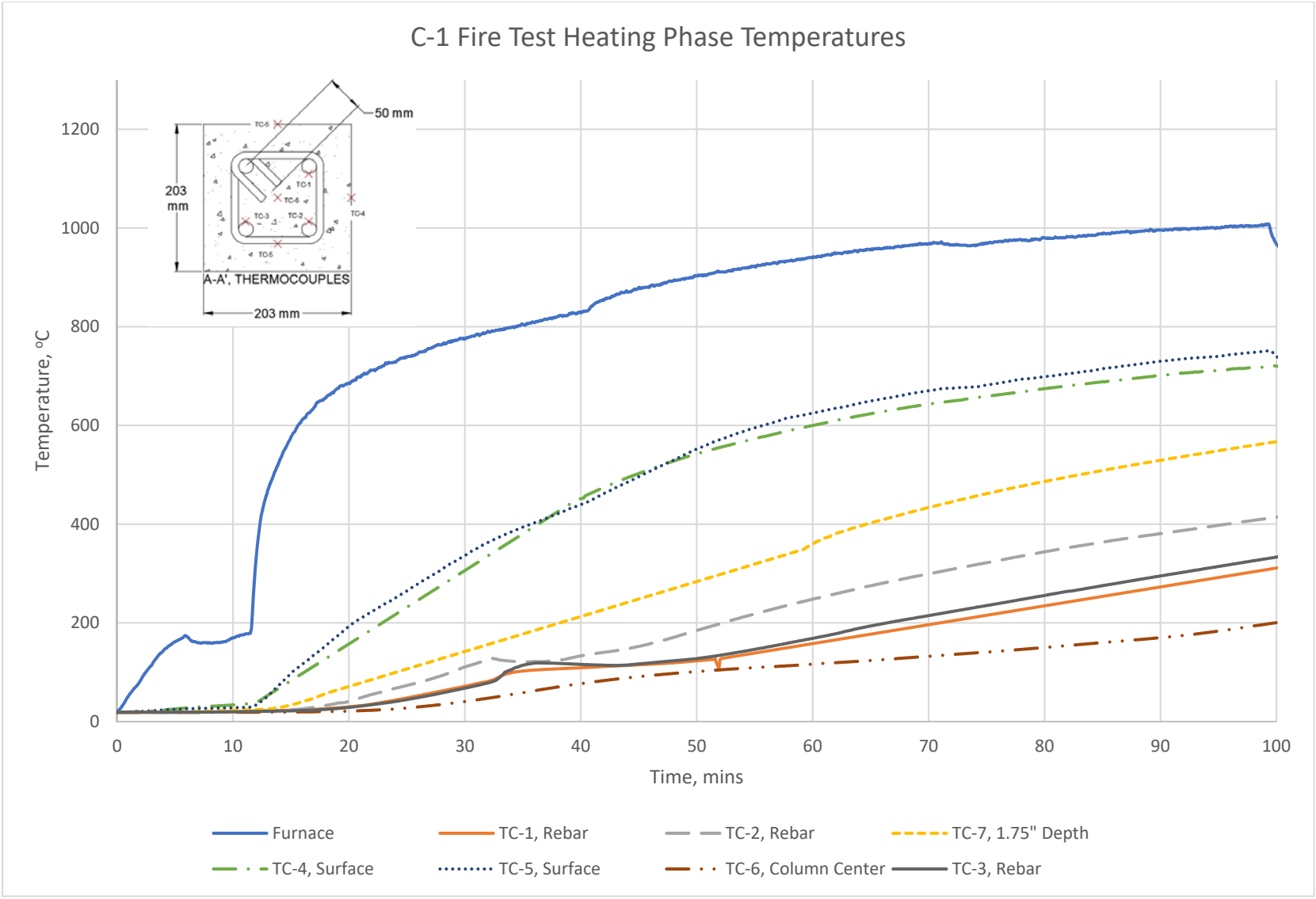

Figure 3.11. Thermal Response in Column C-1 During Heating with Thermocouple Layout Derived from Raut [48].

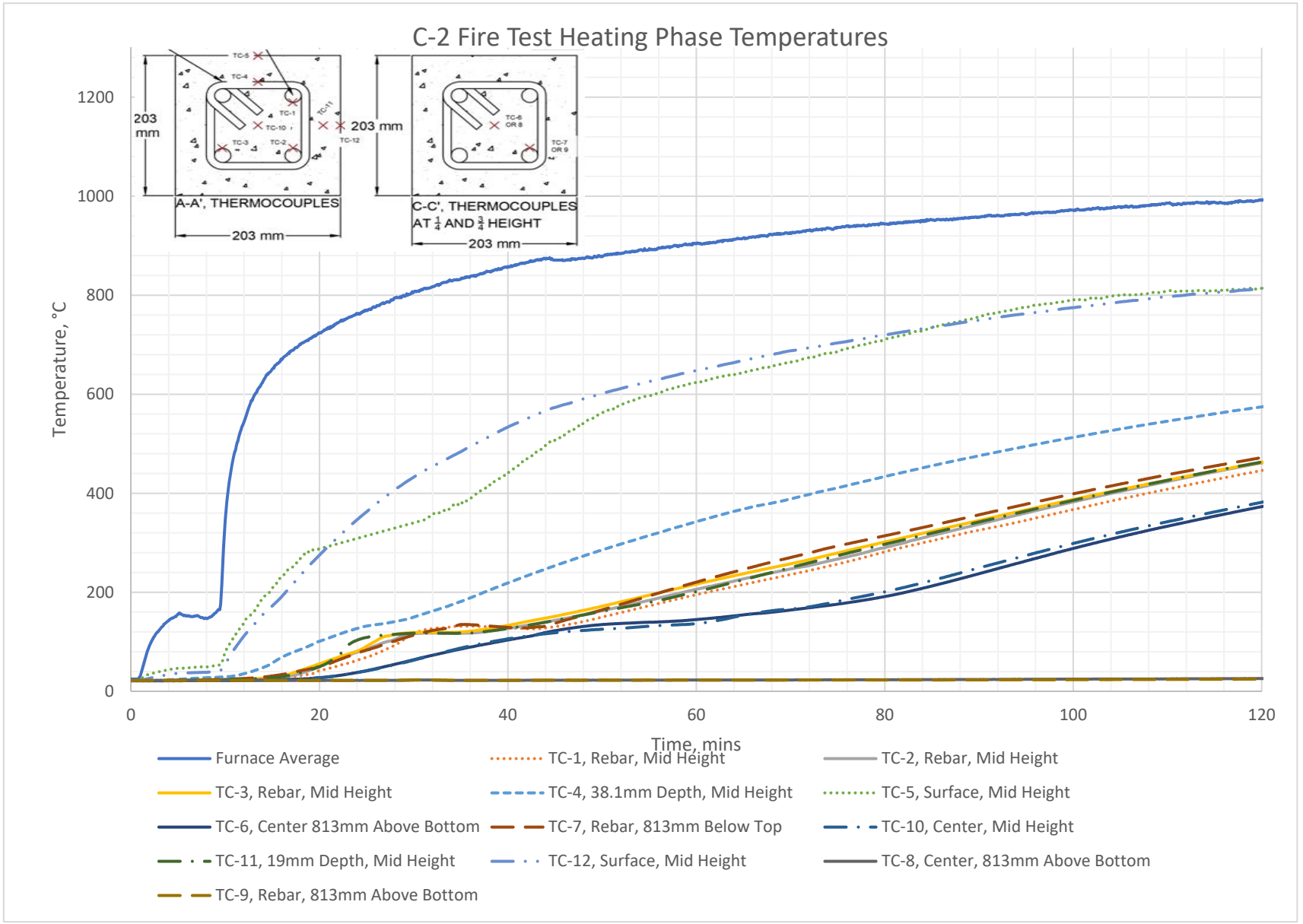

Figure 3.12. Thermal Response in Column C-2 During Heating.

3.3.1.2. Structural Response

The structural response of columns C-1 and C-2 during fire exposure can be determined by plotting measured axial deformation for the two tested columns as a function of fire exposure time (see Figure 3.13). Similar trends in axial expansion are seen in both columns C-1 and C-2 during the early stages of fire exposure (for the first 20 minutes). During this early stage of fire exposure, the rise in deformation (expansion) was mainly due to the thermal gradients developed within the column cross section. As fire exposure time progresses further, temperature within the column cross section begins to rise and the effect of thermal gradients is less pronounced. The axial expansion in each column continues to increase, but at a relatively gradual rate, owing to temperature induced degradation in mechanical properties, especially elastic modulus, of the reinforcing steel and concrete once temperatures increased beyond 400ºC. The rate of axial deformation reduces and eventually plateaus once the furnace temperatures begin to decrease for each of the fire exposures to columns C-1 and C-2.

Axial deformation in column C-1 increases further after about 180 minutes beyond the conclusion of the heating phase. In the case of column C-2, axial expansion continues up to about 200 minutes after the heating phase has stopped. The continued expansion of each column after heating has ceased can be attributed to the thermal lag that occurs during the cooling phase of the specimens. Temperatures inside of the cross section of each column continues to rise after the heating phase of fire has stopped, after 90 and 120 minutes. Furthermore, the longer duration of expansion after the heating phase has stopped in the case of C-2 as compared to that of C-1 is attributed to a longer fire exposure for column C-2.

The maximum deformation was measured to be 9.3 mm and 6.8 mm for column C-1 and C-2, respectively. Interestingly, column C-1 experienced greater axial deformation than that of column

C-2, which was exposed to a more severe fire condition. This can likely be attributed to column C-2 having higher cylinder compressive strength of concrete on the day of fire testing, as well as higher yield stress and tensile stress of the reinforcing steel than that of the concrete and steel in column C-1.

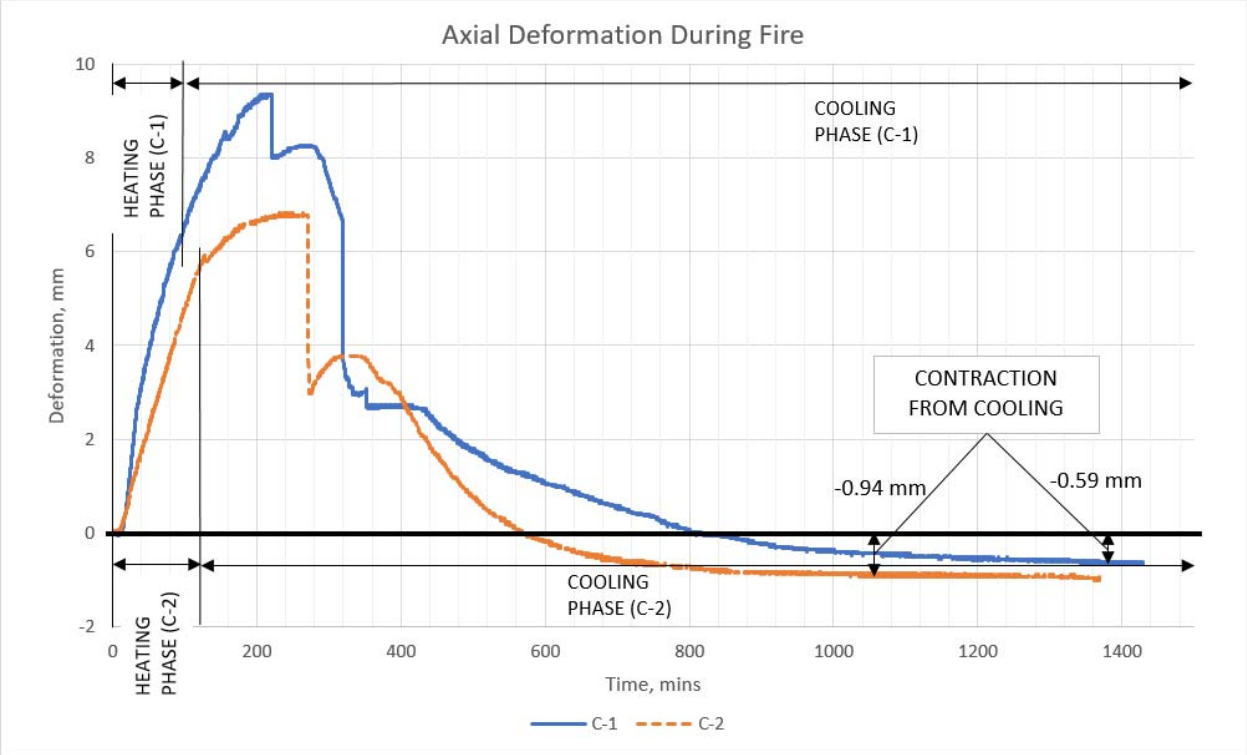

Figure 3.13. Axial Deformation of Columns During Fire Exposure.

The strain in the longitudinal reinforcement of the columns was monitored during fire exposure using high temperature strain gages. The strain gage locations can be seen in Figure 3.2 and Figure 3.3, section B-B'. The measured strain in columns C-1 and C-2 is presented in Figure 3.14 and 3.15, respectively. Strain is plotted as a function of fire exposure time.

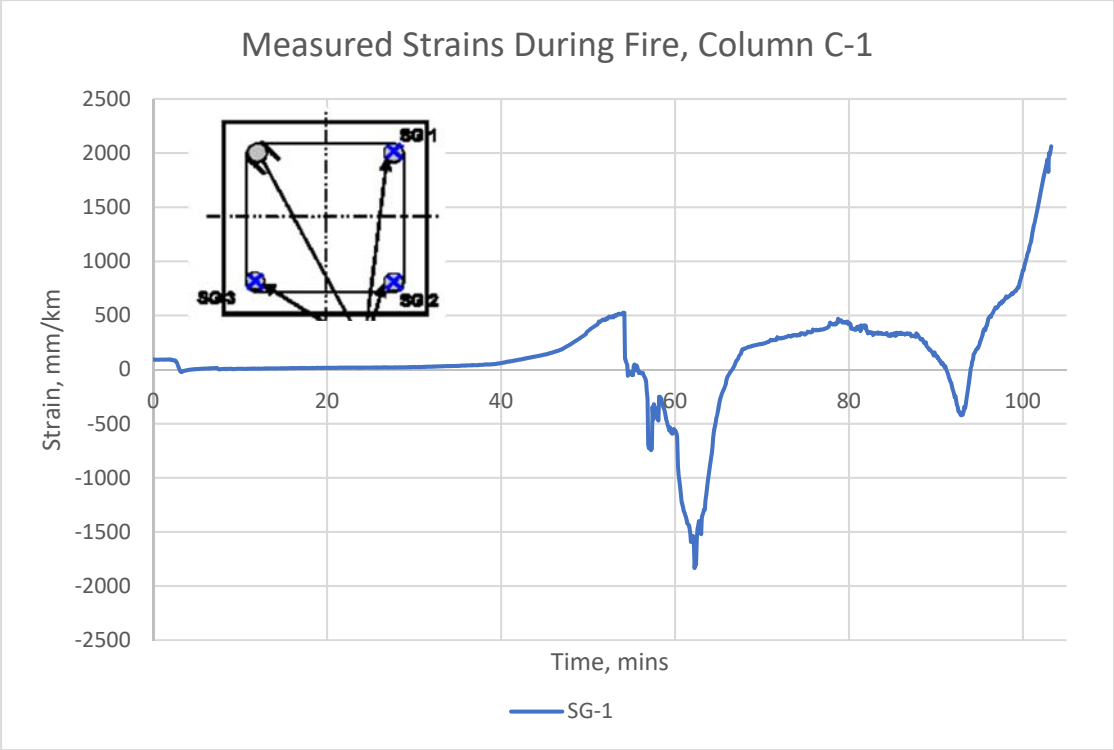

Figure 3.14. Strain Measurements in Column C-1 as a Function of Fire Exposure Time; Strain Gage Layout Derived from Raut [48].

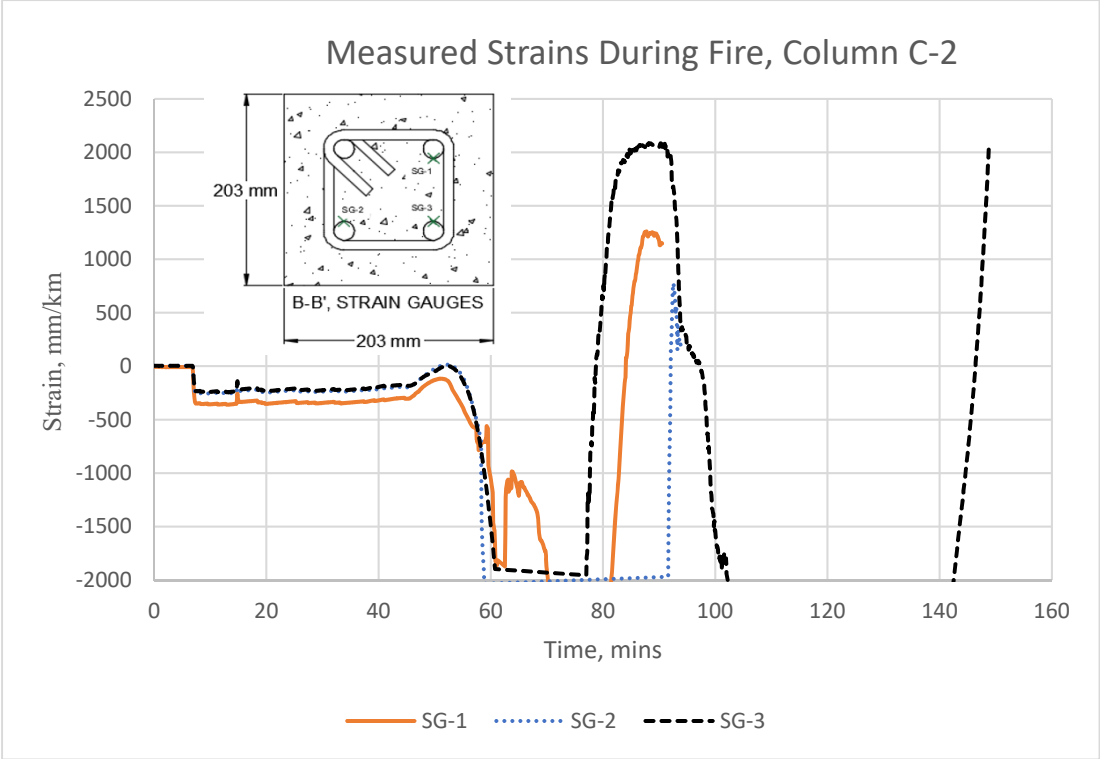

Figure 3.15. Strain Measurements in Column C-2, as a Function of Fire Exposure Time.

The strains measured for columns C-1 and C-2, in Figure 3.14 and 3.15, respectively, seem to be recording nonsensical measurements. This is likely attributed to the cracking of concrete near the which could damage the strain gage, or de-bonding of the strain gage from the reinforcing steel due to the increase in rebar temperature during exposure to fire. Large concrete cracks can lead to direct fire exposure to the strain gage. Only one strain gage (SG-1) of the three shown for column C-1 could record data during the fire test.

It should be noted that at due to the many complex physical and chemical processes that occur at elevated temperatures makes the high temperature strain gages generally unreliable [17]. Previous researchers have attributed the problem to the electrical interference form the operation of the high voltage ignition and the control systems for the furnace [53]. One other reason for this problem could be caused by the high rate of heating experienced by the rebars (about 10^{o}C per minute) in the fire exposure. The strain gages were designed for a lower rate of heating. Due to lack of consistency, the strain gage data collected cannot be definitively used to infer any reliable strain trends in the tested RC columns.

3.3.1.3. Spalling Pattern

The extent of spalling during the fire tests was monitored by making visual observations through the window ports on the furnace and through post-test observations after cooling down of columns. The extent of spalling on the tested columns was classified as minor, moderate, and severe, as given in Table 3.2. Minor spalling is when only small chunks of concrete peels-off of the surface, from outer layers or corners of the RC column. Moderate spalling is when the extent of spalling does not reach the steel reinforcement, while severe spalling is when the reinforcement is exposed directly to fire.

Column C-1 did not exhibit moderate or severe spalling during the application of DF-1, either in heating or cooling phases. After fire exposure and completion of the cooling phase, the column was observed to have had minor spalling with peeling of the outer surface at the corners of the column. It is not clear whether the peeling of the corners happened during the heating or cooling phases. An example of this can be seen in Figure 3.16. Vertical cracks at the supports of column C-1, outside of the furnace, were observed at about 80 minutes into fire exposure. An example of this can be seen in Figure 3.17.

Column C-2 did not exhibit spalling during fire exposure. A negligible amount of corner peeling was observed on the column after completion of the cooling phase. An example of this can be seen in Figure 3.18. Cracking at the supports of column C-2 was not observed during fire exposure.

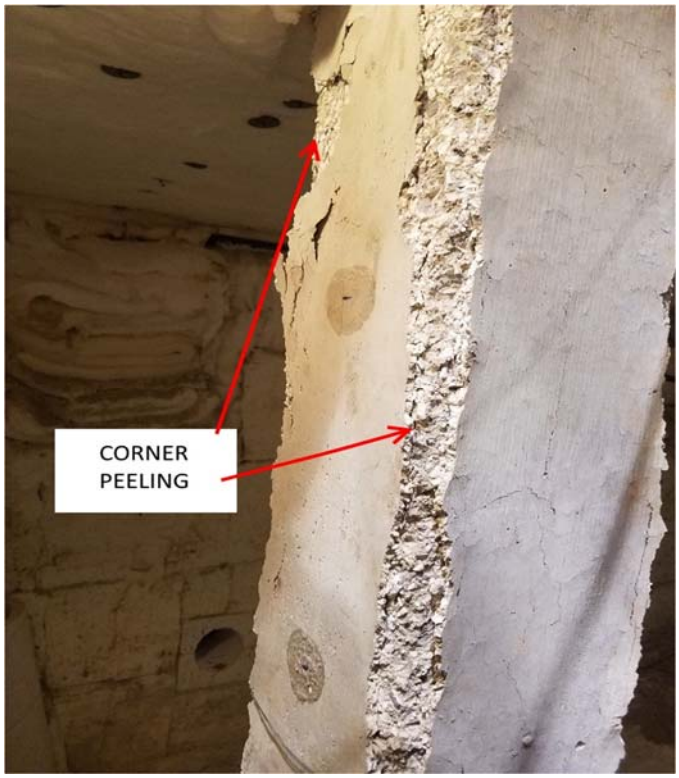

Figure 3.16. Corner Peeling of Column C-1, 40 Hours after Fire Exposure.

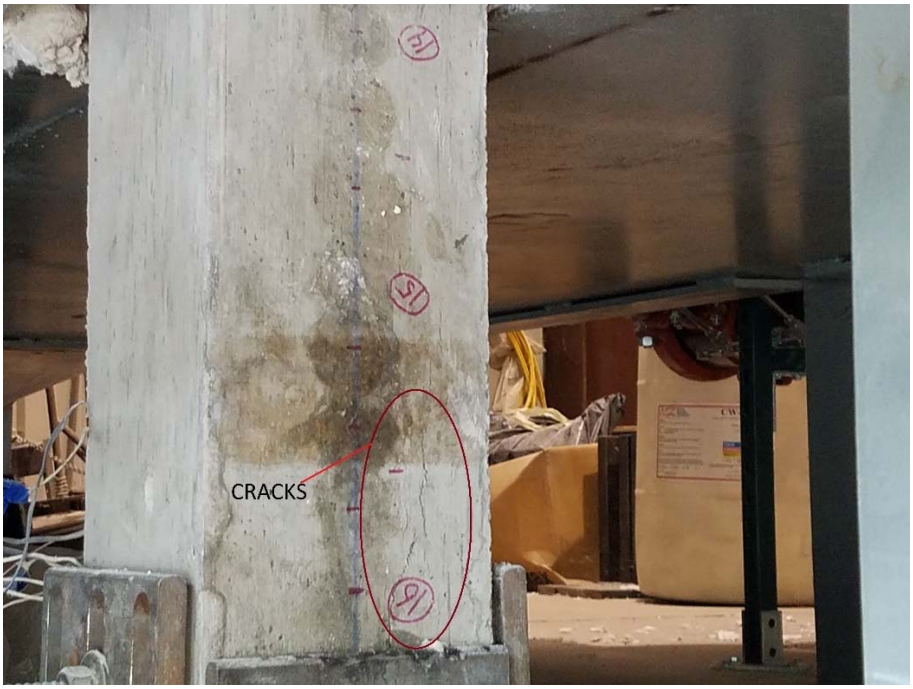

Figure 3.17. Vertical cracking at Bottom of Column C-1 During Fire Exposure; Approximately 80 Minutes into Exposure.

Figure 3.18. Column C-2, 40 Hours after Fire Exposure.

3.3.1.4. Fire Resistance

The fire resistance of the two tested columns is presented in Table 3.2. The time to reach failure is defined as the fire resistance and failure is said to occur when the capacity of the column decreases to a level at which the column cannot sustain the applied load. The tested columns did not experience failure during the heating and cooling phases of the fire exposure.

Table 3.2. Summary of Test Parameters and Results for Fire Resistance Testing.

Column Designation	Test #	Fire Exposure	Concrete Strength (MPa)		Applied Load		Relative Humidity (Test Day, %)	Fire Resistance (mins)	Extent of Spalling (Qualitative)
			28 day	Test day	kN	% of factored ACI			
C-1	I	DF-1	39	49	355	50	59.3	No Failure	Minor
C-2	II	DF-2	46	62.1	400	55	70.0	No Failure	Minor

3.3.2 Behavior During Cooling Phase

After fire exposure, the columns were allowed to cool down in the furnace to ambient temperature through a controlled cooling phase (see Figure 3.7). Once cooled, each column was left in the fire testing chamber for a period of 48 hours. Due to the different heating durations in fire exposures DF-1 and DF-2, the time taken for each of the column to cool to ambient conditions were different. This in turn will have an impact on the residual deformations, extent of late stage spalling and continuation of the degradation of residual capacity in each column.

3.3.2.1. Thermal Response

The temperatures at various locations in the cross section of each column was monitored up to 24 hours after the heating phase of the fire exposure. It can be observed that the temperatures recorded near the exposed surface of each column start to decrease as soon as the fire temperature enters the decay phase. The inner layers of concrete, however, continue to see a sustained increase in temperature even after fire temperature begins to decay (see Figure 3.19 and Figure 3.20). This lag in temperatures can be seen with the high thermal inertia of concrete.

Rebar temperatures in column C-1 were observed to attain a peak value of nearly 550°C, which occurred approximately 90 minutes after completion of the heating phase. Likewise, the center of

column C-1 experienced a peak temperature value of 474ºC occurring approximately 120 minutes after the completion of the heating phase. In the case of column C-2, rebar temperatures reached a peak value of nearly 600ºC occurring 80 minutes after the heating phase of DF-2 has completed. Furthermore, the temperature of concrete in the center of the column reached a peak value of about 580ºC after 110 minutes into the decay phase of the fire exposure.

The time taken by each column to attain ambient temperature appears to be governed by the duration of the heating phase. For the sake of comparison, the rebar and column center temperatures experienced in column C-2 occurs nearly 600 minutes after the start of fire exposure are nearly twice that of the temperatures in column C-1. Additionally, the temperatures in the center of each column is over 120ºC occurring 540 minutes (9 hours) after the start of fire exposure.

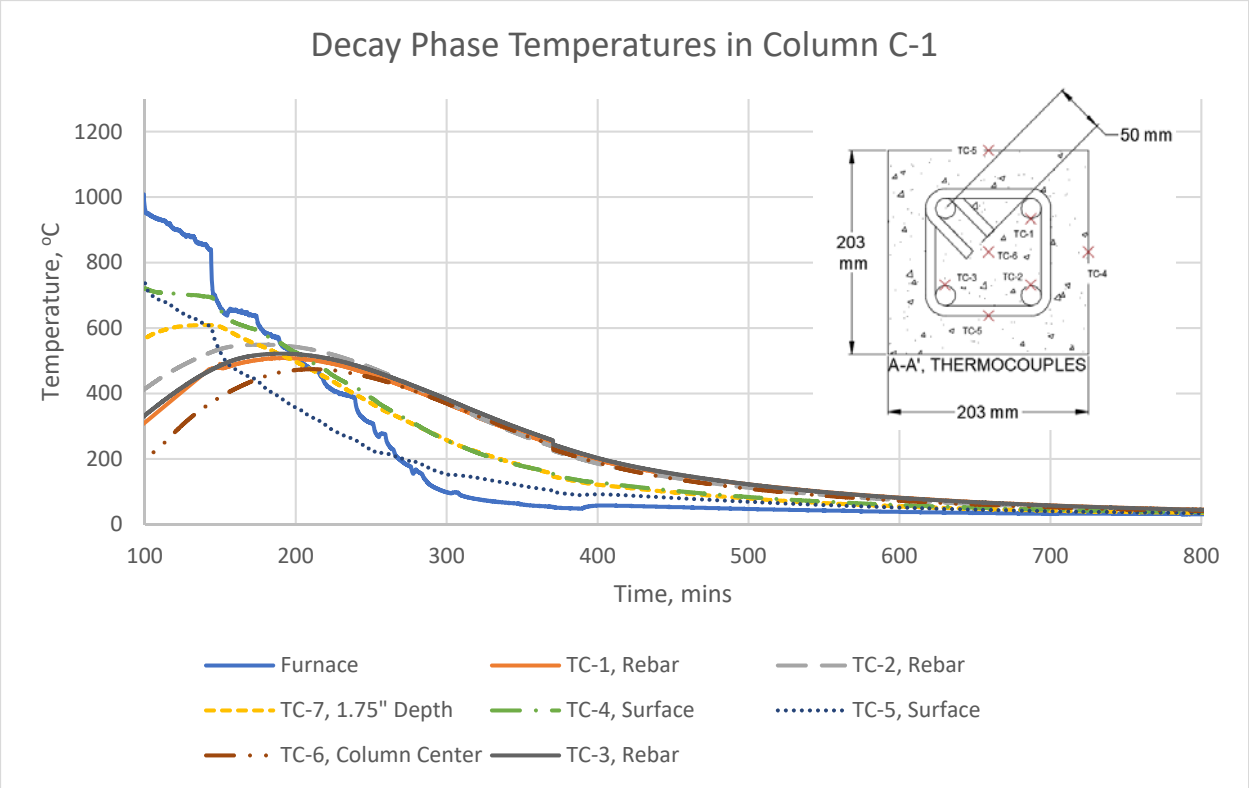

Figure 3.19. Decay Phase Temperatures in Column C-1; Thermocouple Layout Derived from Raut [44].

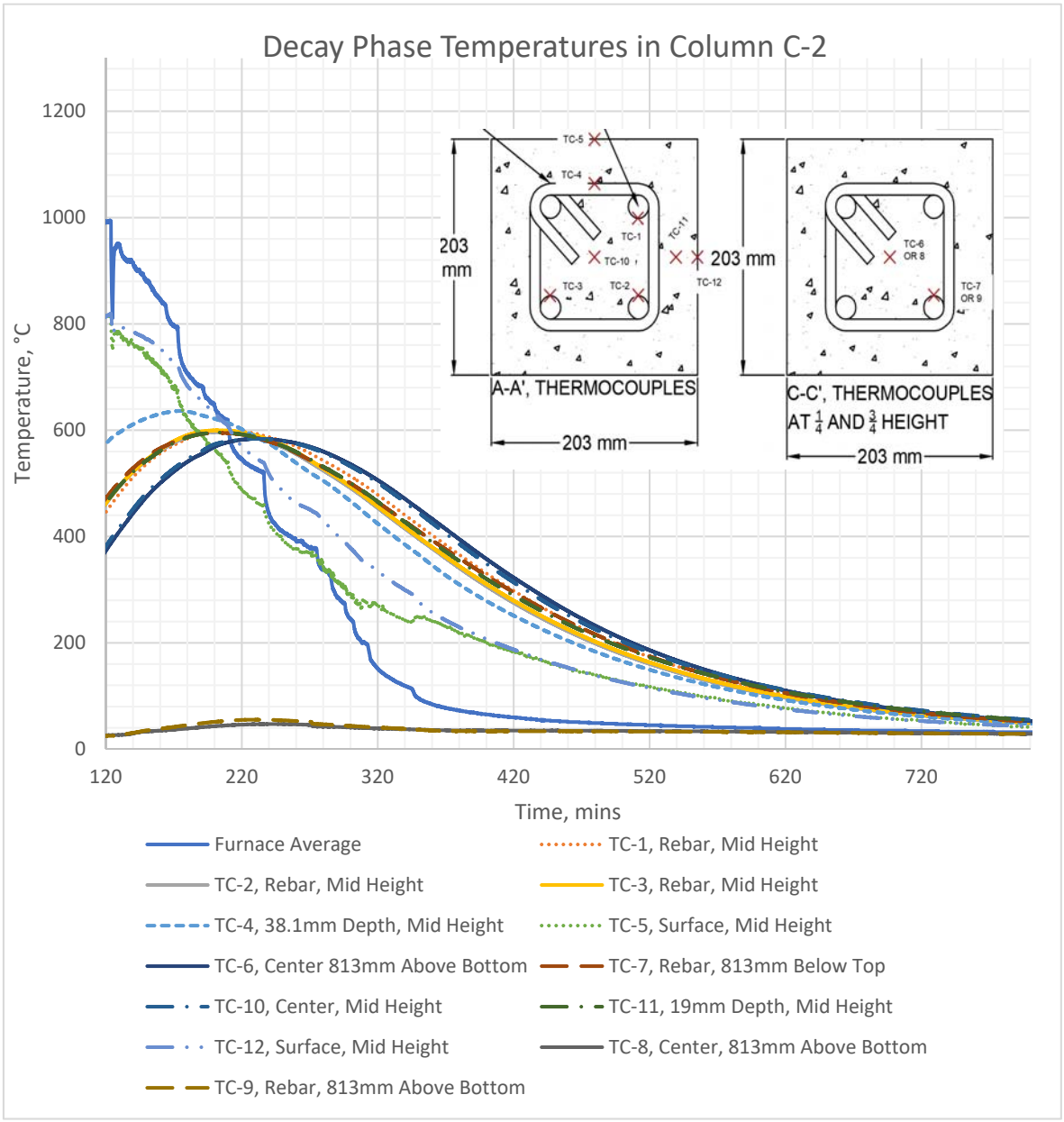

Figure 3.20. Decay Phase Temperatures in Column C-2.

3.3.2.2. Structural Response

The axial deformations of columns C-1 and C-2 during the cooling phase of the column is plotted in Figure 3.13. The axial deformation appears to be primarily governed by temperatures experienced within the compression rebar. For column C-1, the axial expansion continues to

increase until about 280 minutes after the start of fire exposure, as the rebar temperature stays above 400ºC. Similarly, the axial expansion of column C-2 continues to increase until about 360 minutes after the start of fire exposure, as the rebar temperatures stay above 400ºC. The deformations that occur during this stage of fire exposure, in which rebar temperatures are maintained above 400ºC, is attributed to high temperature creep that is typically observed in the later stages of fire exposure to NSC structural members.

Axial deformations in each column begins to decrease once the rebar temperatures in the column begin to decrease below 400ºC. The recovery of deformation is attributed to the increasing strength and stiffness with decrease column and rebar temperatures. It should be noted that the rate of recovery of deformation in each column is nearly three times slower than its rate of increase during heating. This phenomenon is likely caused by the nonlinear nature of the cooling phase which causes the temperatures in the concrete sections to reduce gradually as well as the thermal inertia of concrete, which allows the member to retain heat for a prolonged period. Furthermore, irreversible changes occur in the thermal and mechanical properties of concrete during the heating phase which results in a marked difference in the response of the columns during the cooling phase. Once each column reaches room temperature, the residual deformation it attains does not reduce further. This is caused by the irreversible temperature induced damage in concrete and reinforcing steel, as well as the residual plastic stresses and strains that exist in the columns, even after the cool to room temperature. The residual deformations were measured to be 0.5 mm and 1 mm, in columns C-1 and C-2, respectively. While the difference in peak rebar temperatures experienced in column C-2 was about ten percent greater than that of C-1, the residual deformation in column C-2 is about 50% greater than what was observed in column C-1. This implies that the residual deformations that occur in fire exposed RC columns are more sensitive to the intensity of the fire

scenario rather than the rebar temperatures themselves. Furthermore, each column was not observed to exhibit failure during fire exposure, and were tested for residual capacity after cooling to ambient conditions.

3.3.2.3. Late Stage Spalling

Observations were made through a viewport every few minutes to gauge the extent of spalling in the columns during cooling phase of fire. For each column, no noticeable spalling occurred, even after 120 minutes of fire exposure for the case of column C-2. Nonetheless, corner peeling of column C-1 was observed when the beam cooled to ambient conditions. The observed corner peeling occurred during the cooling phase of the fire test. Column C-2 experienced a negligible amount of corner peeling.

3.3.3 Residual Response of Fire Exposed Columns

The residual capacity of each column (C-1 and C-2) was evaluated once the columns cooled to ambient conditions. The columns were incrementally loaded to failure after cooling to ambient conditions. The load-deformation response, progression of cracking and failure modes were monitored during residual testing.

3.3.3.1 Non-Destructive Evaluation

Non-destructive testing was carried out on columns C-1 and C-2, prior to performing the residual load carrying capacity tests. UPV measurements were taken both before and after fire exposure. The pulse travel time as well as pulse velocity are each plotted as functions of distance, for each of the tested specimens. Pulse velocity and pulse travel time for each column is compared by plotting the pre-fire and post-fire trends on the same plots. The pulse travel time and pulse velocity prior to fire exposure is relatively consistent, due to the specimens being in an undamaged state.

The sudden changes of slope observed in the pulse velocity and pulse travel time are attributed to the presence of natural shrinkage cracks in the undamaged specimens.

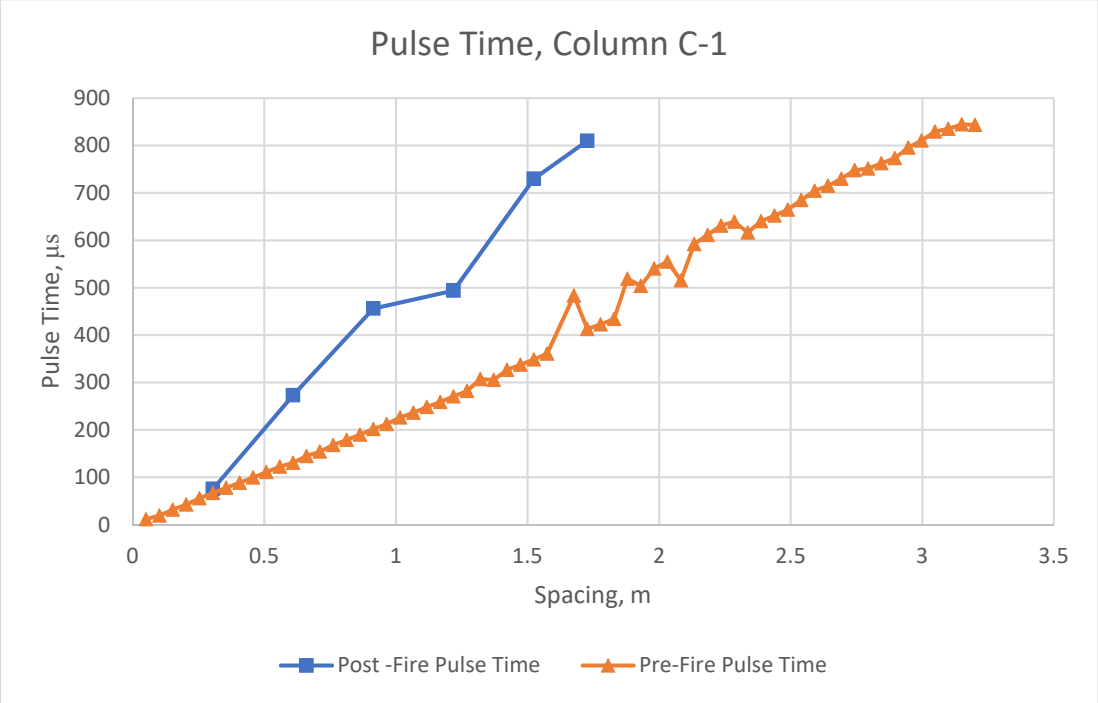

Figure 3.21. Pre- and Post-Fire Pulse Travel Time in Column C-1.

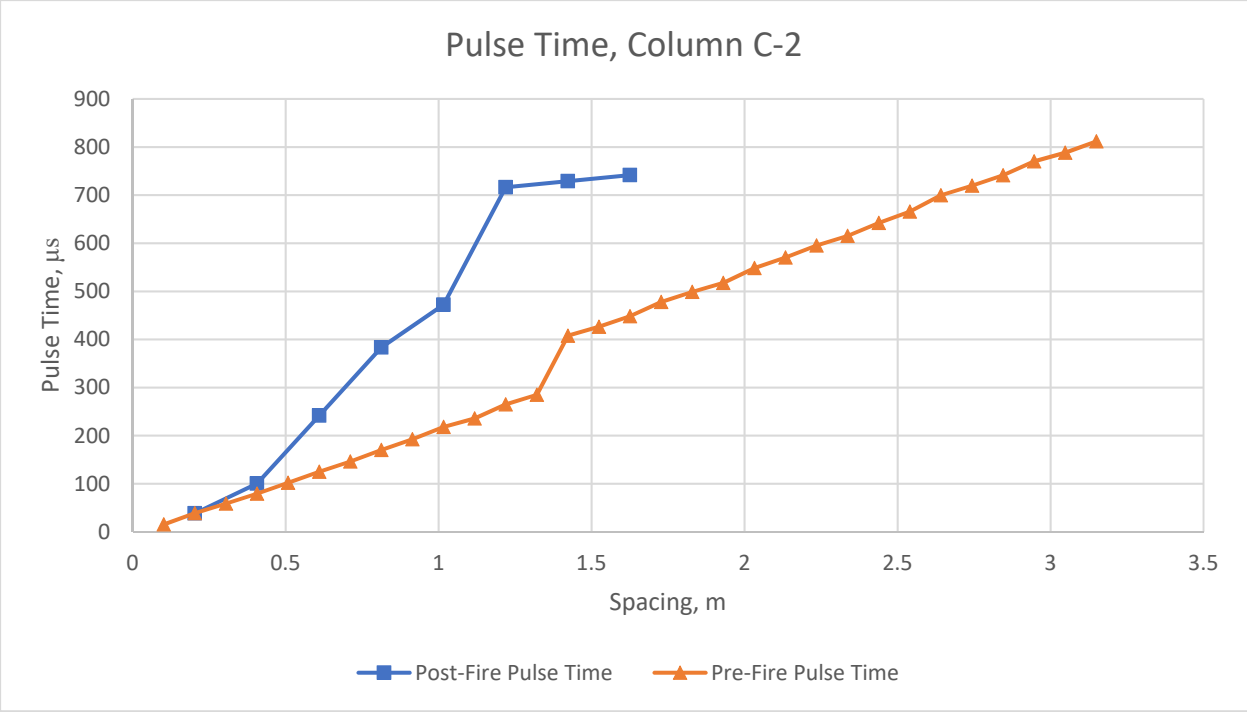

Figure 3.22. Pre- and Post-Fire Pulse Travel Time in Column C-2.

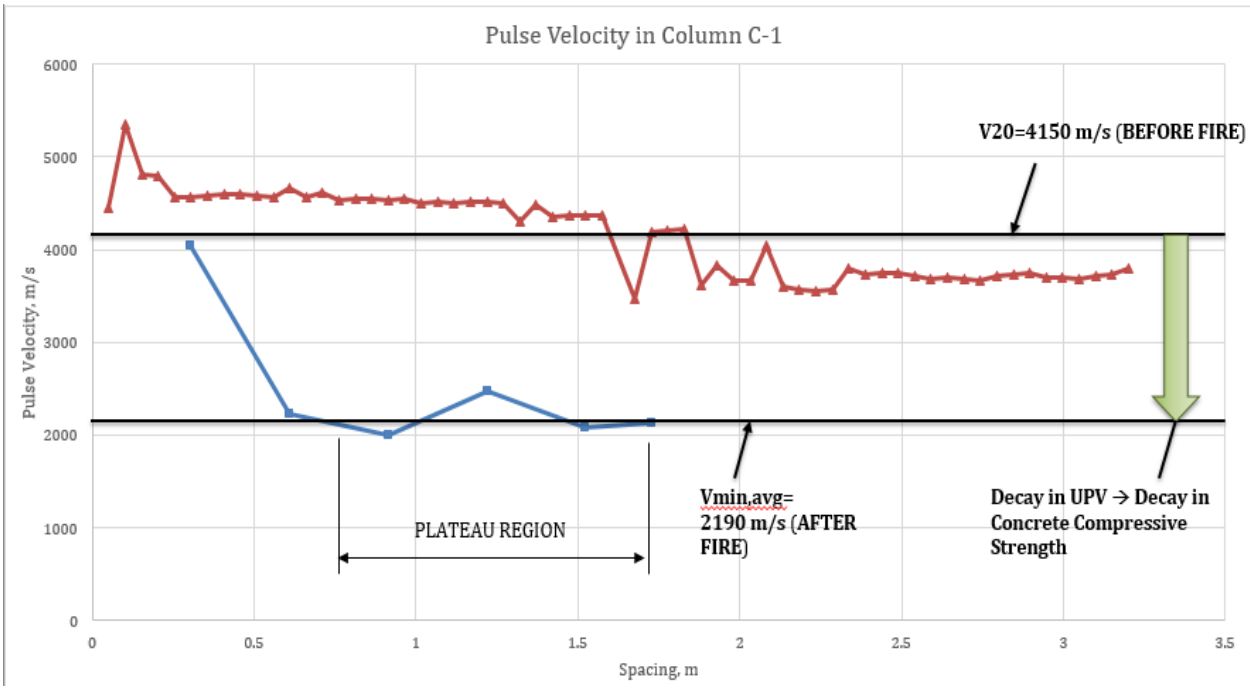

Figure 3.23. Pre- and Post-Fire Pulse Velocity in Column C-1.

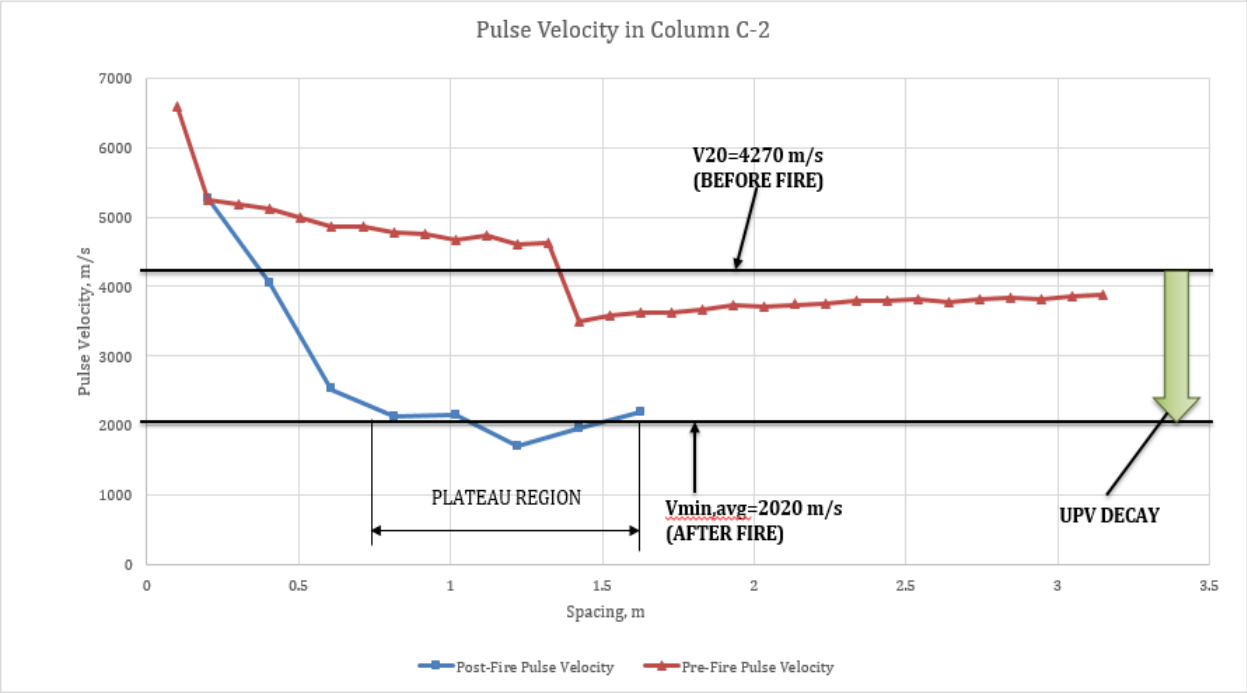

Figure 3.24. Pre- and Post-Fire Pulse Velocity in Column C-2.

In each of Figure 3.23 and Figure 3.24, the listed value of V_{20} corresponds to the average pulse velocity of each specimen prior to fire testing. The value V_{min} corresponds to the average pulse velocity of the fire damaged specimen in the region where velocity values plateau. The value of V_{min} corresponds to the pulse velocity of the damaged layer of concrete due to fire exposure. Pulse velocity decay due to fire exposure was found to be 47.2% and 52.7% for columns C-1 and C-2, respectively. This in turn led to an estimated concrete compressive strength decay of 35.0% and 47.0% for columns C-1 and C-2, respectively. The compressive strength decay values were obtained from Figure 3.8. A summary of these findings can be observed in Table 3.3.

Table 3.3. Concrete Compressive Strength Decay due to Fire Damage.

Specimen ID	V_{20} (m/s)	V_{min} (m/s)	% UPV Decay	% Strength Decay	Test Day Strength, MPa	Damaged Layer Strength, MPa
C-1	4150	2190	47.2%	35.0%	49	17.2
C-2	4270	2020	52.7%	47.0%	62.1	29.2

3.3.3.2 Structural Response

During residual testing phase, axial deformation of each column was measured through LVDT's installed in the same plane as the top of column. The load deformation plot for columns C-1 and C-2 is plotted in Figure 3.25. Each column exhibits three distinct stages in deformation progression i.e., linear elastic response, inelastic response and softening of reinforcing steel.

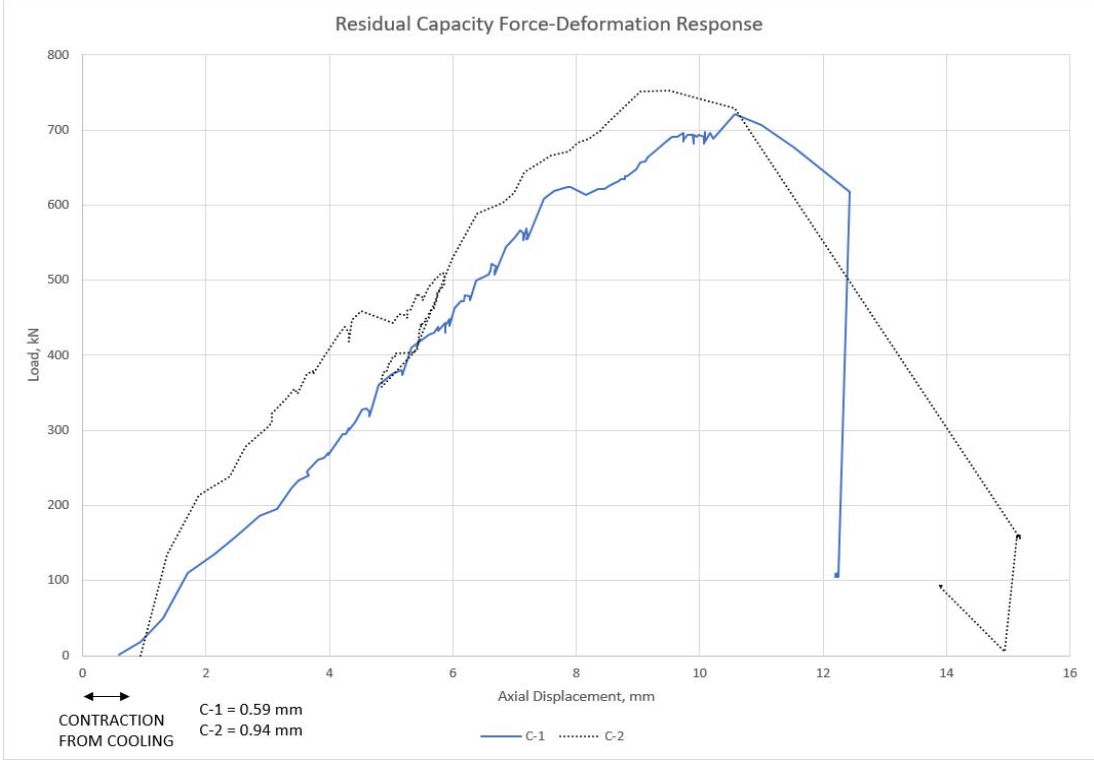

Figure 3.25. Residual Load Deformation Response of C-1 and C-2.

In the first stage, the load deformation response of the fire damaged columns C-1 and C-2 follows a fairly linear trend, until the onset of yielding in steel reinforcement. The load at yielding was approximately 110 kN and 210 kN for columns C-1 and C-2, respectively. In the second stage, an inelastic hardening response is observed. This is due to the concrete cover over the reinforcing steel beginning to break off, and more load being carried by the reinforcing steel as well as the concrete that the steel is confining. The third stage of the load-deformation response is characterized by the onset of softening of the reinforcing steel up to the point of failure. In this stage, the column deformation continues to increase while the applied load simultaneously drops. This is primarily due to the crushing of concrete in addition to the softening of the reinforcing steel. The onset of failure appears to occur at a load of approximately 720 kN in column C-1. The failure load of column C-2, in which rebar temperatures were around 600^{o}C, was observed to occur at 750 kN, which is 20% *greater* than that observed in column C-1. The higher residual load for column C-2, which experienced a longer heating duration than that of column C-1, is attributed to having a nearly 25% higher room temperature compressive strength.

The peak load carrying capacity was found to be 720 kN and 752 kN for columns C-1 and C-2, respectively. Each of the measured residual capacities are very close to the factored room temperature capacity of 732 kN, computed as per ACI 318 [2] (see Appendix II for detailed calculations of column capacity). This is despite the occurrence of the fire-induced damage in the columns. The higher capacity at room temperature can be attributed to the strain hardening of the steel reinforcement (which is conservatively not accounted for in the ACI 318 design equations for strength [2]), in addition to the higher concrete compressive strength.

To better understand how the residual capacity of each column was affected by fire exposure duration, load level and concrete strength, Table 3.4 is generated. The nominal axial capacity is calculated using the actual test day room temperature compressive strength of concrete. The capacity is calculated from the following (provided by ACI 318 [2]):

$$P_0 = 0.85f'_cA_c + f_sA_S$$

Where:

P_0 =Nominal axial room temperature capacity of RC column

f'_c =Room temperature concrete compressive strength

A_c =Area of concrete

f_s =Yield stress of longitudinal reinforcing steel

A_S =Area of longitudinal reinforcing steel

When comparing the values of the real nominal capacity to that of the measured residual capacity, it becomes obvious that with a more severe fire exposure, the residual capacity of the column is reduced.

Table 3.4. Residual Capacity Comparison.

Column Designation	Test #	Fire Exposure	Concrete Strength (MPa)		ACI 318 Design Capacity (kN)	Nominal (Real) Capacity (kN)	Residual Capacity	
			28 day	Test day			kN	% of Real Nominal Capacity
C-1	I	DF-1	39	49	732	2144	720	33.6%
C-2	II	DF-2	46	62.1	732	2591	752	29.0%

3.3.3.3 Crack Patterns and Failure Modes

The formation of vertical cracks was observed near ultimate failure of each column during the residual capacity testing. These cracks formed only in the fire exposed region of each column. The

failure mode of each column was explosive in nature, in that the concrete burst outward in the central portion of each column. Photographs of each column just after residual capacity testing can be seen in Figure 3.26 and 3.27, for column C-1 and C-2, respectively. Upon removal of each column from the furnace, after residual testing, it was observed that the longitudinal reinforcement experienced some degree of buckling. It is likely to have occurred from the load applied during the residual test. Examples of the longitudinal buckling observed in columns C-1 and C-2 can be viewed in Figure 3.28 and 3.29, respectively.

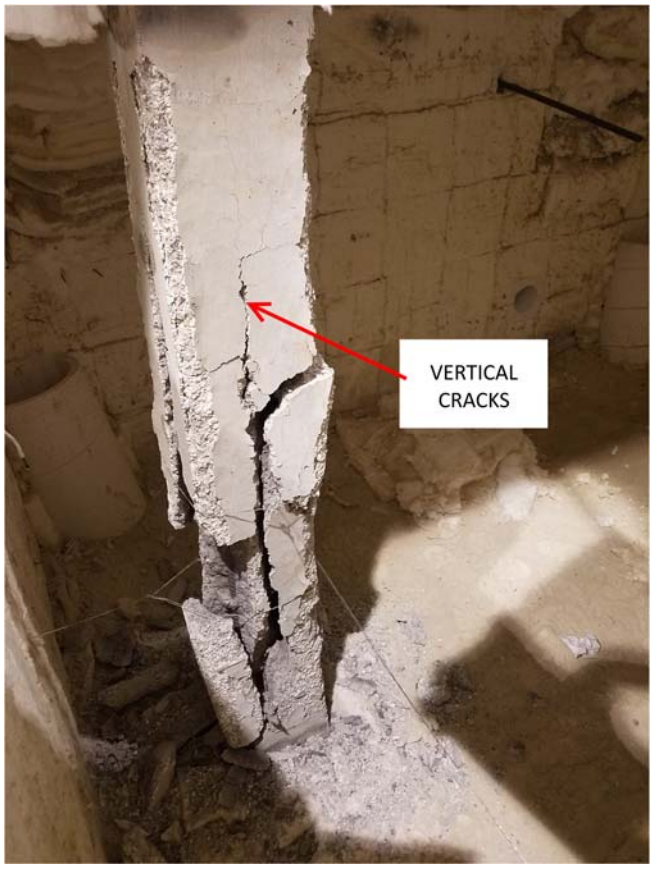

Figure 3.26. Condition of Column C-1, Post Residual Testing.

Figure 3.27. Condition of Column C-2, Post Residual Testing.

Figure 3.28. Buckled Rebar after Residual Testing of column C-1.

Figure 3.29. Buckled Rebar After Residual Testing of column C-2.

3.4 Summary

The presented experimental program includes the fabrication of RC columns made of NSC, exposure of the columns to two different design fire scenarios, post-fire NDT and observations and post-fire residual capacity testing. Each fire exposed column survived fire exposure, with little to no obvious surface damage present. The NDT testing in the form of UPV measurements, however, indicated a significant decrease in the quality of the outer layers of fire exposed concrete. The decrease in UPV is indicative of a reduction of concrete strength in the outer layers.

Moreover, each column retained most of its design capacity, as computed per ACI 318 [2]. This is due to the conservative nature in the design of axially loaded RC members, as well as a relatively short duration of heating in each fire scenario. Additionally, the room temperature compressive strength of concrete measured the day of testing is significantly higher than the specified compressive strength, which will further lead to a greater retention of capacity.

CHAPTER 4

4 NUMERICAL MODEL

4.1 General

Performing fire tests for evaluating residual capacity of structural members can prove to be quite expensive, complex, and time consuming. Development of numerical models to predict the behavior of RC columns both during and after fire exposure is an alternative to large scale experimental studies. As shown in Chapter 2, many numerical studies have been performed to predict the behavior of RC columns during and after fire exposure. The reviewed numerical studies are based mainly on performing a sectional analysis and thus neglects global structural failures arising from uniaxial or biaxial buckling. Therefore, numerical models that are based on a global approach considering the entirety of fire exposed member, are to be developed for tracing the residual capacity of RC columns. The developed models need to account for buckling of the member, nonlinear high temperature material properties, various restraint conditions and realistic failure criteria. To accomplish this, a finite element model was developed in commercially available finite element software, ABAQUS. The adopted analysis procedure involves a three-stage analysis to predict the thermal and structural response of a fire exposed RC column during and after fire exposure. This chapter presents the development of the finite element model for the thermal and structural analyses conducted.

Each stage of analysis is presented in Figure 4.1:

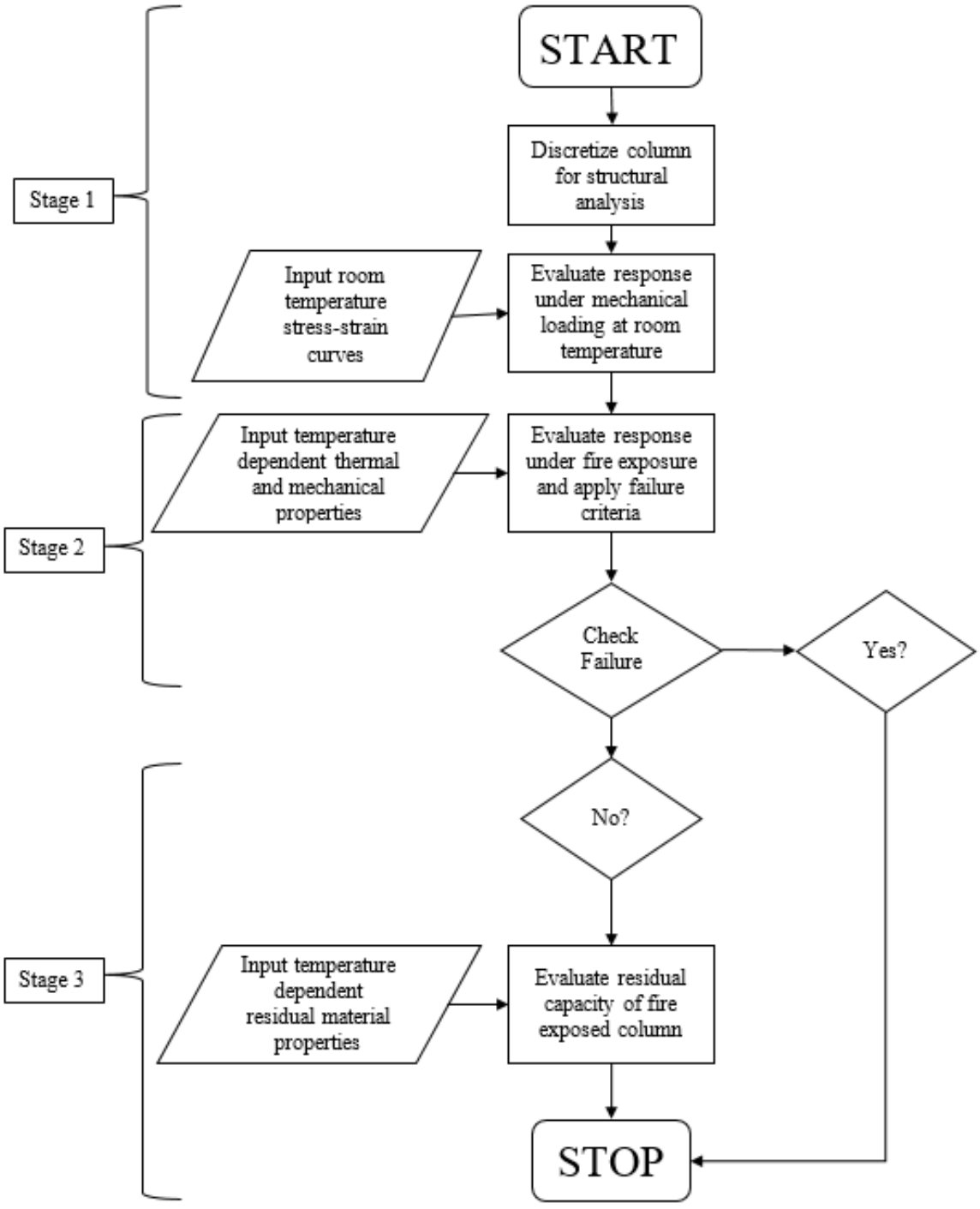

Figure 4.1 Flow Chart Illustrating Steps for Evaluation of Residual Capacity of RC Columns.

4.2 Analysis Procedure

The process to determine the residual response of a fire exposed RC column comprises of three stages, namely:

- Stage 1: Evaluate column capacity at room temperature prior to fire exposure,
- Stage 2: Perform fire resistance analysis of the RC column during fire exposure,
- Stage 3: Determine the post-fire residual response after cooling of the column.

The above analysis procedure can be undertaken in several finite element packages, such as ABAQUS, ANSYS, etc. The flowchart in Figure 4.1 serves as a schematic to the developed analysis procedure, which starts with room temperature axial capacity estimation, followed by realistic fire exposure and ending with residual capacity estimation.

In Stage 1, the capacity of the RC column at room temperature is evaluated through a detailed finite element method by incrementally increasing the load on the structure (column) until failure occurs. Estimation of the ultimate capacity can alternatively be obtained by using the specified strength equations available in design standards, such as ACI 318 [2]. The capacity obtained in the room temperature analysis in Stage 1 is used to determine the applied load level on the column during fire exposure of Stage 2. The Stage 1 capacity is also used to estimate the extent of degradation in capacity obtained from Stage 3 (residual) analysis. Room temperature mechanical properties of steel and concrete are used in the Stage 1 analysis.

The response of the RC column is evaluated in Stage 2 of analysis, in which the member is subjected to a fire exposure, load level and boundary conditions as encountered in a real structural member (column). The realistic loading that is present in a typical fire event are applied to the column prior to the analysis during fire exposure. The fire exposure is applied through a time-temperature curve defined by the user. The thermal load is applied incrementally until the fire

exposure period ends, or the column fails. The response from thermal and structural analysis are recorded at the end of each time increment to check the state of the RC column under different failure limit states. Temperature dependent thermal and mechanical properties of concrete and reinforcing steel, which are different for heating or cooling phases of fire exposure, are to be input into ABAQUS.

As seen in the schematic of analysis in Figure 4.1, Stage 3 analysis is carried out at the completion of cooling in Stage 2. There are residual stresses and strains in the fire exposed column after cooling caused by the accumulation of damage that was induced in the column due to heat, load level and material properties. This state of the column is the initial conditions for the residual analysis in Stage 3.

In the third stage of analysis, the cooled RC column is incrementally loaded until failure. The structural response of the column during loading is recorded at each load step. The residual capacity corresponds to the maximum load in which the column can carry prior to failure. For this stage of analysis, the residual material properties of concrete and reinforcing steel are required.

4.2.1 Analysis Parameters

The post fire residual capacity of RC beams is dependent upon load level, fire exposure scenario, structural parameters and material characteristics both during fire and after cooling [36]. Therefore, it is reasonable to assume that these same parameters are likely influencing the residual capacity of RC columns as well. Higher load levels coupled with a more severe fire exposure can lead to a column losing a significant portion of its cross section in the form of spalling. This in turn will lead to a lower residual capacity after fire exposure. However, if a loss of cross section resulting from spalling is not evident, a minor decrease in capacity is possible. In each of these

cases, the residual capacity of a fire exposed RC column is necessary to determine the best course for reuse and retrofitting fire damaged member.

The post fire material properties of both concrete and reinforcing steel can be significantly influenced by the maximum temperature the rebar and concrete experienced during fire, the time allowed for cooling after fire, as well as the method of cooling used (such as natural air cooling or water quenching, for example). Residual compressive strength of concrete that experienced temperatures in excess of 220ºC can decrease by up to 20% of its original room temperature strength immediately after cooling down [49]. Nevertheless, it has been reported that with sufficient recovery time at room temperature, concrete can regain up to 100% of its original room temperature compressive strength [39]. Additionally, recent studies suggest that the 'short term' or 'temporary phase' where the compressive strength of concrete does not recover, can last up to three years after fire exposure [52]. Following the 'short term' recovery phase, there is a 'long term' response of RC structures following fire exposure in the form of a significant recovery in concrete strength. The cooling method (air cooled or water quenched) does not influence the post-fire compressive strength of concrete as significantly as it does the failure strain and elastic modulus [43]. Concrete that is cooled by water quenching will generally attain a higher compressive strain at failure than it would have, had it been cooling in air. There is no significant change in the post-fire failure strain of concrete as compared to its original room temperature failure strain for exposure temperatures up less than 220ºC [13].

The residual strength of reinforcing steel following fire exposure primarily depends on the maximum temperature that was reached in the steel reinforcement. If the temperature of hot-rolled reinforcing steel does not exceed 500ºC, rebars will recover nearly 100% of its initial room temperature yield strength upon cooling [45]. However, if reinforcing steel temperatures exceed

500ºC, it will regain only part of its original room temperature strength. The ratio between ultimate strength and yield strength is approximately 1.5 at room temperature decreases with increasing temperatures and becomes 1.0 at around 800ºC [32]. This implies that strain hardening of steel is less likely at very high temperatures.

Therefore, selection of fire exposure scenario and the strength recovery assumed in both reinforcing steel and concrete after fire exposure are critical factors in evaluated the residual capacity of fire exposed RC columns.

4.2.2 Failure Criteria

In each stage of performing the residual strength analysis of a RC column, several different failure criteria are to be applied. The strength limit state where failure occurs when the concrete or rebar reaches their ultimate strain, governs failure in the Stage 1 of analysis. In Stage 2 of analysis (during fire exposure), the RC column will experience high temperatures which leads to reduced capacity owed to the degradation of strength and stiffness properties of both concrete and reinforcing steel. Failure of the column in Stage 2 is based on the strength limit state, where the column is said to have failed when the degrading capacity of the column falls below that of the applied load. When the applied load exceeds the columns capacity, a rapid increase of axial deformations is observed. The duration to the point at which the applied loading has exceeded the capacity of the column represents the fire resistance of the member.

In residual capacity analysis (Stage 3), the residual material strength as well as residual deformations will generally govern the failure of the column. Residual capacity is defined as the maximum applied load just prior to a rapid increase in axial deformation.

4.3 Development of Finite Element Model

The above procedure is employed for evaluation of the residual capacity of a fire exposed RC column. A finite element model is developed to evaluate the response of the column before fire exposure (Stage 1), during fire exposure (Stage 2), and after fire exposure (Stage 3).

4.3.1 General

Each stage of analysis is carried out using the finite element computer program ABAQUS. Constitutive models for concrete and reinforcing steel are defined within the software package and the modeling of the fire exposed RC column is undertaking using a sequentially coupled thermo-mechanical analysis procedure. With this procedure, the mechanical analysis uses the results generated in the heat transfer analysis (Stage 2), however, no reverse dependency exists.

Specifically, two sets of discretized models are used to analyze the behavior of the RC column during fire exposure in Stage 2 of analysis; one for the thermal analysis and the other for mechanical (strength) analysis. Results from the thermal analysis are applied along the RC column as thermal loads in the structural model. The temperature dependent mechanical properties of concrete and reinforcing steel are incorporated in the structural model.

It should also be noted that the analysis can be divided into sequential steps in ABAQUS with the response state (such as stresses, strains and temperature) of the column occurring in each step. This software feature allows for the response parameters to be transferred from Stage 2 (fire exposure) to Stage 3 (residual capacity) of analysis. Furthermore, a load-controlled method of analysis is utilized.

4.3.2 Modeling Assumptions

The following assumptions are made in the development of the finite element model:

- Bond between steel in concrete is assumed to be perfect and that the strain in the reinforcement is equal to that of concrete.
- The spalling of concrete has not been explicitly modeled in the analysis, which implies that the developed model is applicable only for normal strength concrete (NSC) with compressive strengths of 70 MPa or lower. The susceptibility of spalling in concrete members with strengths less than 70 MPa has shown to be minimal [34].
- The mechanical properties of concrete in the cooling phase are only considered in the "short-term" period after the member has cooled to ambient conditions. Recovery of concrete compressive strength that occurs months after fire exposure is conservatively ignored in the analysis.
- The thermal conductivity, heat capacity and thermal expansion of both concrete and reinforcing steel has been assumed to be entirely reversible. The heat capacity of concrete that reduces from the loss of moisture during heating, as well as the thermal expansion (or shrinkage) of concrete in the cooling phase are both neglected for the sake of simplicity.

4.3.3 Discretization of the Column

In order to carry the three stages of analysis, two sub-models have been developed for the thermal and structural analyses. The structural sub-model is necessary to carry out the strength analysis in Stages 1, 2 and 3 of analysis. The thermal sub-model is required to carry out the thermal analysis which undertakes the heat transfer calculations to compute the nodal temperatures within the RC column.

For thermal analysis, concrete is discretized by using DC3D8 elements, which are eight node linear brick elements. The reinforcing steel is discretized by using DC1D2 elements, which are two-node link elements. Each are available in the ABAQUS library, and have nodal temperatures (NT11) as the only active degree of freedom.

For the structural analysis, the RC column was discretized using eight-node continuum elements (C3D8) and two-node link elements (T3D2) for concrete and reinforcing steel, respectively. The continuum elements used to discretize the RC have three translational degrees of freedom. This element can be used for 3D modeling of solids with or without reinforcement and is capable of accounting for the cracking of concrete in tension, crushing in compression, creep and large strains [16]. The two-node elements used to model the one-dimensional reinforcing bars are assumed to deform by either axial stretching or shortening. Each element is connected by "pin" joints at their nodes and only translational displacements at each node are used in the discretization. When strains are in the elements for reinforcing steel are large, the formulation is simplified by assuming that the elements are made of an incompressible material. This approach has been used to effectively model reinforcement explicitly where nodes of reinforcement are coincident with the corresponding nodes of concrete [16]. The perfect bond between concrete and reinforcing steel is achieved by using the embedded region constraint, which is a feature built into ABAQUS.

While the use of beam elements is thought to be more computationally efficient for both structural and thermal models [16], three dimensional elements are required for prediction of an accurate temperature distribution within a RC member. Moreover, similar modeling strategies have worked well for researchers in the past [22]. Use of three dimensional elements will also allow for future modeling complex phenomena, such as spalling of concrete.

The temperature transfer from concrete to reinforcing steel is accomplished by use of a tie constraint. Elements at the face of the discretized column are used to simulate the surface effect of convection and radiation from four-sided fire exposure to the member.

4.3.4 Input Parameters for Analysis

Parameters such as boundary conditions, loading, fire scenario and material properties are required to carry out the different stages of analysis. For each analysis stage, these critical input parameters used in the analysis are presented.

4.3.4.1. Stage 1: Room Temperature Capacity

In determination of room temperature axial capacity, room temperature stress-strain relationships are used for both concrete and reinforcing steel as defined in Eurocode 2 and 3 [20,12] (See Appendix I). Concrete is assumed to be linear elastic until the stress levels reach 33% of the compressive strength of concrete (f'c). Following the linear portion of the stress strain curve of concrete, a parabolic non-linear increase in stress, which is representative of the development of microcracks in concrete, is used. A linear descending branch of the stress-strain relation in concrete is used to define the post-peak softening behavior of concrete. Moreover, built-in package of a damaged plasticity constitutive relation is used to model the complex behavior of concrete, which involves different failure mechanisms such as crushing or cracking of concrete. The reinforcing steel uses a metal plasticity model which employs the von Mises yield surface with associated plastic flow and isotropic hardening, which are available in ABAQUS [16].

4.3.4.2. Stage 2: Fire Exposure Analysis

In the heating stage, room temperature dependent thermal and mechanical properties of reinforcing steel and concrete are assumed to follow that of Eurocodes 2, 3, and 4 [20,12,10]. Additionally, the variation of Poisson's ratio of concrete is assumed based on Elghazouli and Izzueddin (2001) [18]. The temperature dependent thermal and mechanical properties of concrete and reinforcing steel are different in the cooling phase than they are during the heating phase. Therefore, a linear interpolation between the elevated and residual material properties after cooling is used based on Eurocodes 1 and 2 [20,11]. Additionally, the recommended Eurocode 2 values for convective and radiative heat transfer coefficients for concrete are taken to be 25 $Wm^{-2o}C^{-1}$ and 0.8 $Wm^{-2o}C^{-1}$, respectively [20].

4.3.4.3. Stage 3: Residual Capacity Analysis

After cooling of the RC column has completed, the residual compressive strength of concrete is assumed to be 10% less than the compressive strength at the maximum temperature in the fire exposure stage of analysis (Stage 2) (see Figure 4.3). This is based on the Eurocode 4 (2005) [10] recommendations that has shown to yield accurate results in the prediction of residual capacity of RC columns immediately after cooling to ambient conditions [10]. The residual stress-strain relationship for reinforcing steel is calculated using the degradation trends reported by Neves et. al (1996) [45] (Figure 4.2).

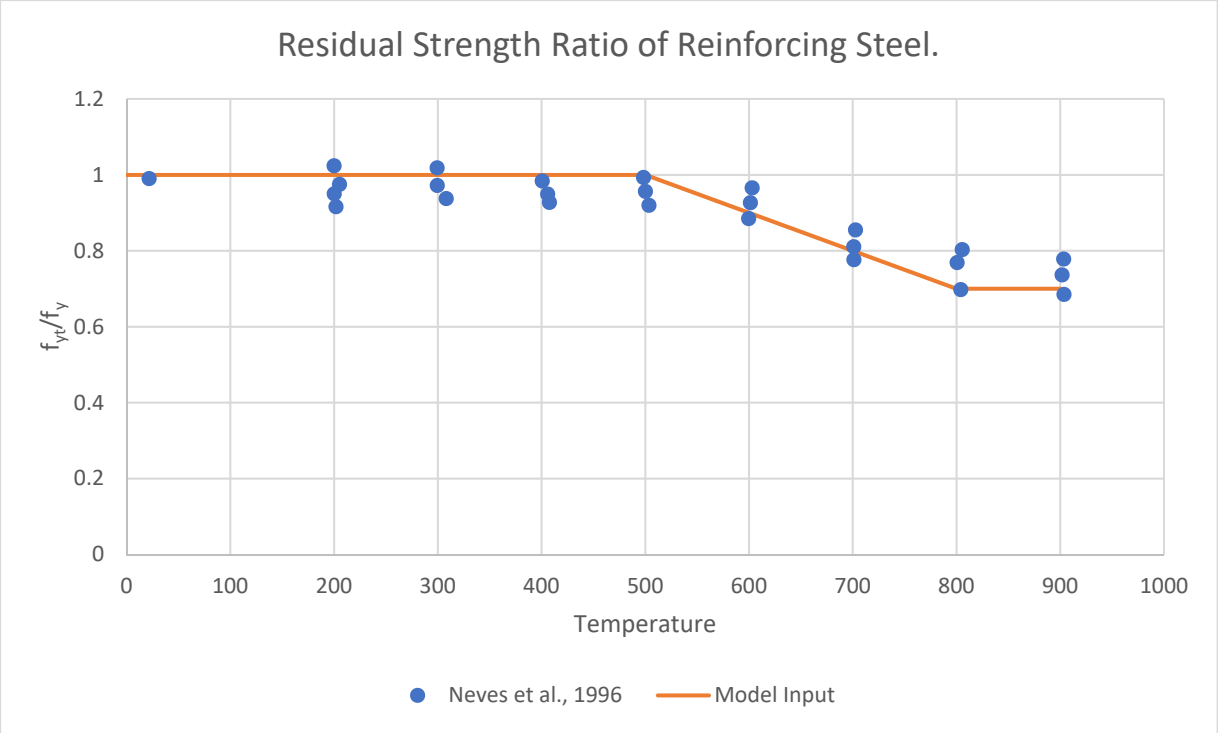

Figure 4.2. Residual Strength Ratio of Reinforcing Steel, derived from Neves, et al. (1996) [41].

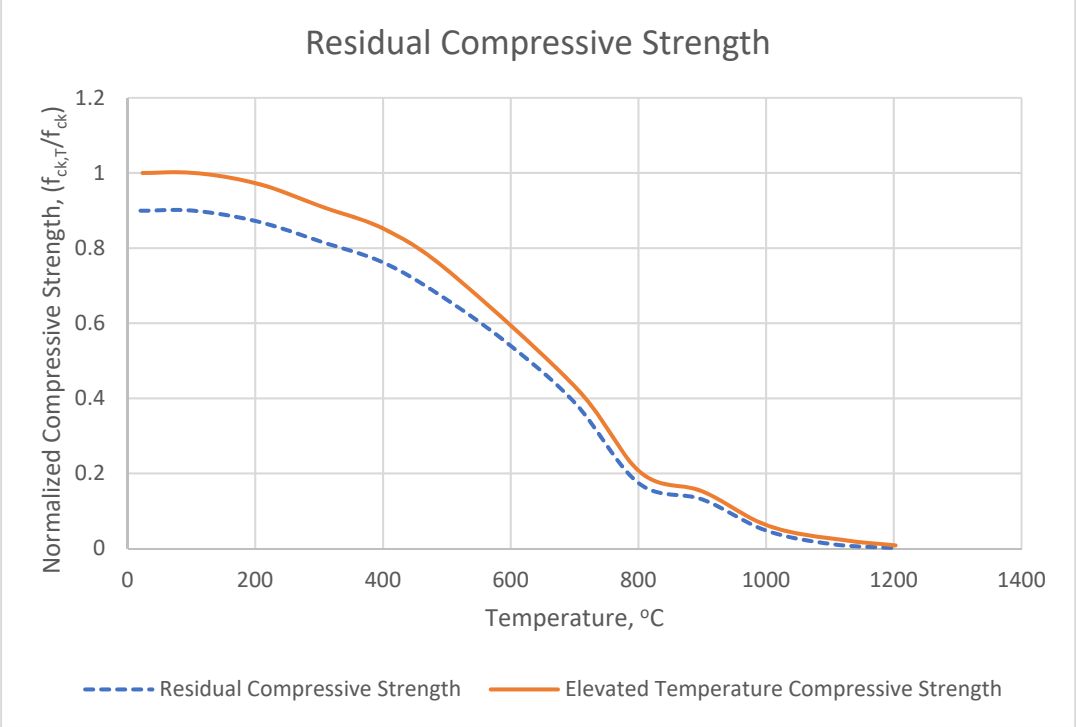

Figure 4.3. Normalized Compressive Strength of Concrete with Maximum Exposure Temperature; adapted from Kodur and Agrawal (2016) [32].

4.4 Output Results

The primary output variables that are generated during the three stages of analysis are axial deformations, stresses and temperature fields. In room temperature capacity analysis, failure is said to occur when any further increment in the applied load leads to an instability of the column or crushing of the concrete. During the fire exposure stage of analysis, temperatures are applied to each node at specific time steps in order evaluate the structural response of the RC column under fire exposure. The subroutine UFIELD, provided in ABAQUS framework [16], is used to determine if either concrete or reinforcing steel is under heating or cooling and applies the appropriate material properties for structural analysis. In the residual capacity analysis (Stage 3), the load deformation response is used to evaluate the residual load carrying capacity of the fire exposed RC column. The capacity at failure is calculated by integrating stresses experienced within the concrete and reinforcing steel, as generated in ABAQUS [16].

4.5 Model Validation

The developed finite element model is validated against the observed experimental behavior of columns. Thermal prediction of the fire exposed RC columns are compared to the experimental thermal response in the column center as well as longitudinal reinforcement. The structural response during fire exposure is validated by comparing the observed and predicted axial deformations during fire. Finally, the validation of the residual capacity is presented by comparing the predicted and observed load-deformation response of each of the fire exposed columns.

4.5.1 Thermal Response

The predicted and actual thermal response of each of columns C-1 and C-2 are presented in Figure 4.4 and 4.5, respectively. It can be seen that the predicted temperature values of the column center and rebar are quite a bit higher than the actual thermal response in columns C-1 and C-2. This is

attributed to the calculated temperature dependent thermal properties based on EC2 [20] and EC4 [12] being conservative in nature. It can also be explained by the nature of the finite element method in typically predicting a 'stiffer' response as compared to actual behavior. Further, column C-1 has a higher temperature difference between the experimental column centers and rebar temperatures. This can be explained by the thermal conductivity significantly changing with respect to age of the column. Nonetheless, the trend of the predictive values matches closely to the experimental values, and is considered to reasonably predict the actual thermal behavior of a fire exposed RC column.

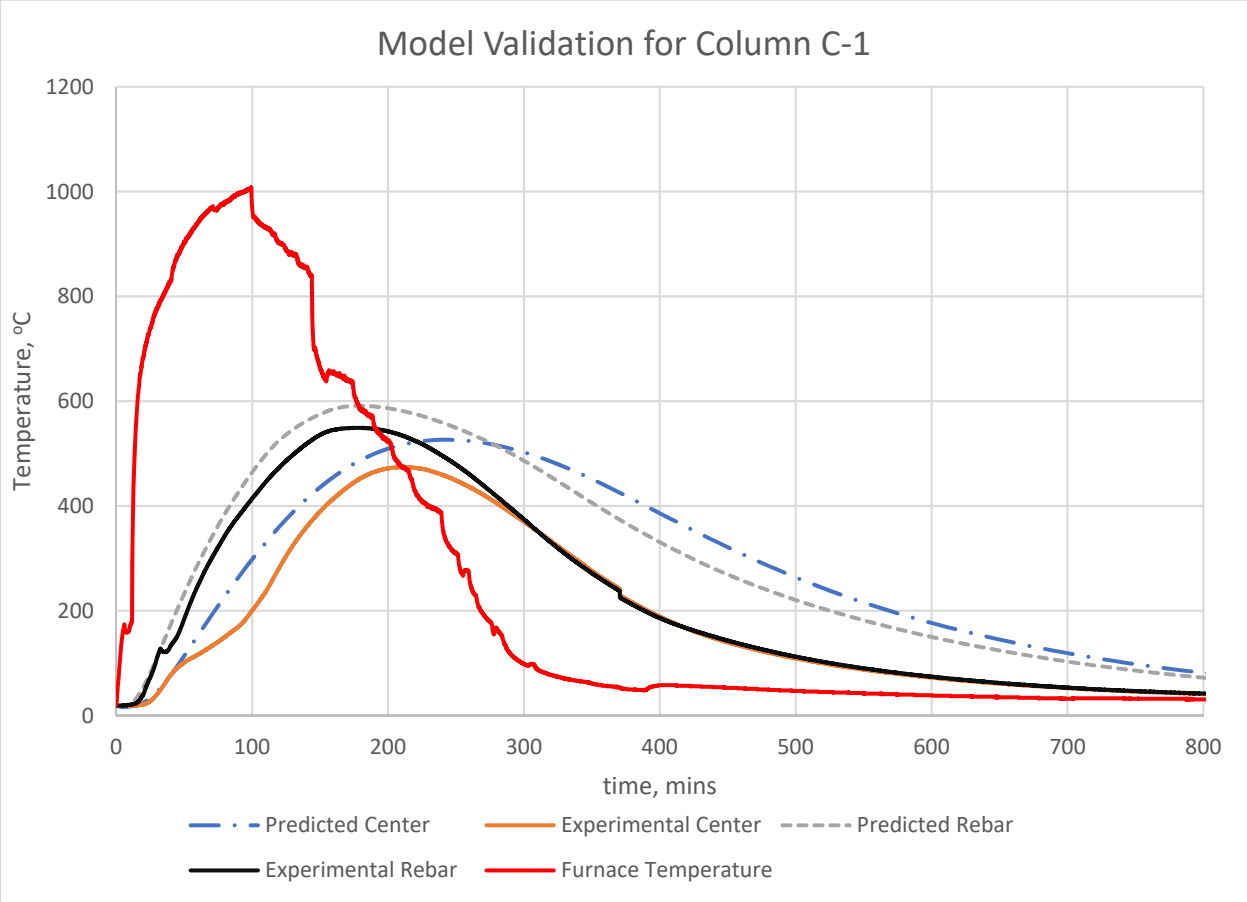

Figure 4.4. Thermal Model Validation for Column C-1.

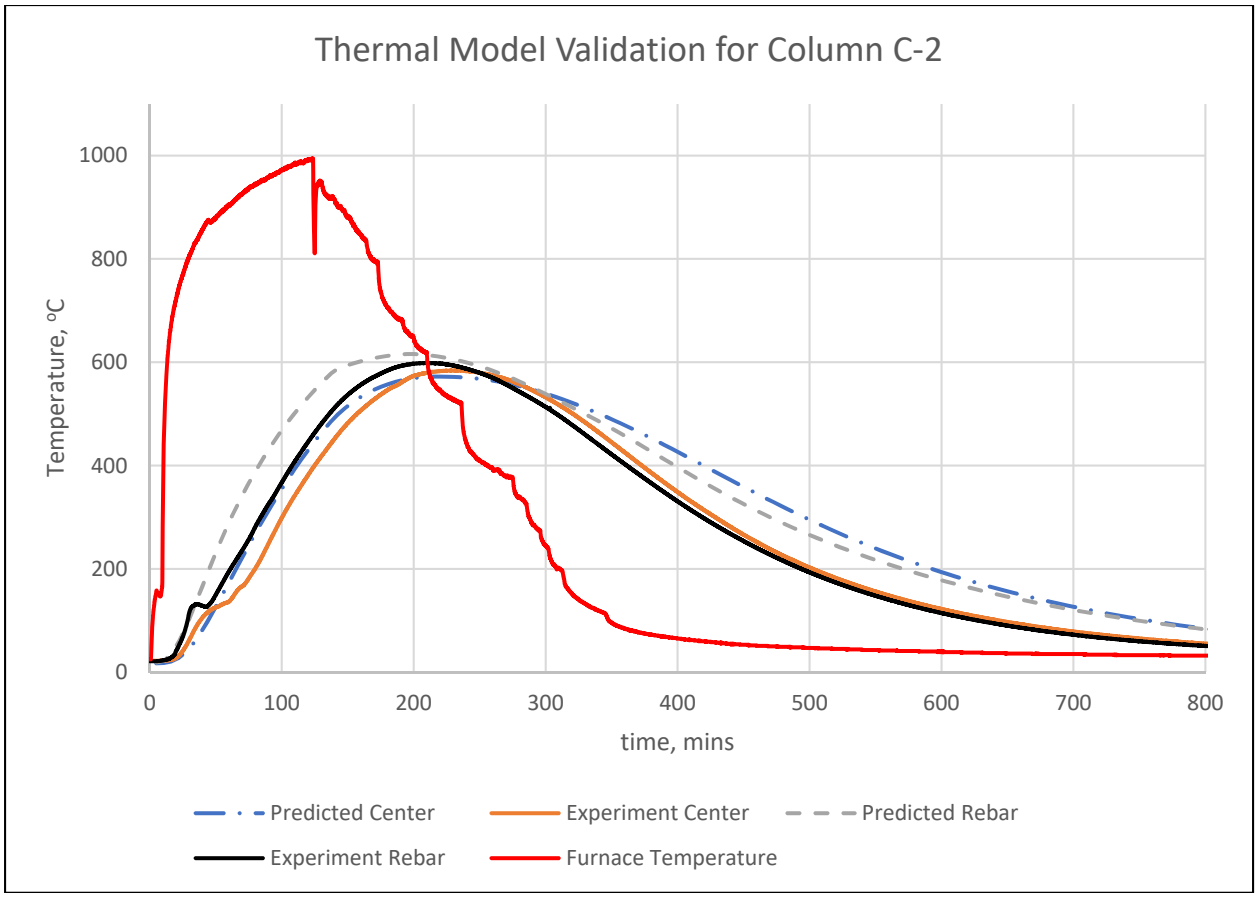

Figure 4.5. Thermal Model Validation for Column C-2.

4.5.2 Structural Response During Fire

The predicted and actual axial deformation of columns C-1 and C-2 are plotted as a function of fire exposure time in Figure 4.6. The deformations predicted in the FE model were slightly higher than the observed deformations from the experiments. This is attributed to the temperatures predicted in the modeled column during the heating phase to be higher than the observed experimental values. Consequently, there is greater thermal expansion in the analysis and hence greater axial deformations. In the cooling phase, however, the contraction predicted by the numerical model is lower than experimental values as load induced thermal strain is not explicitly considered in the analysis.

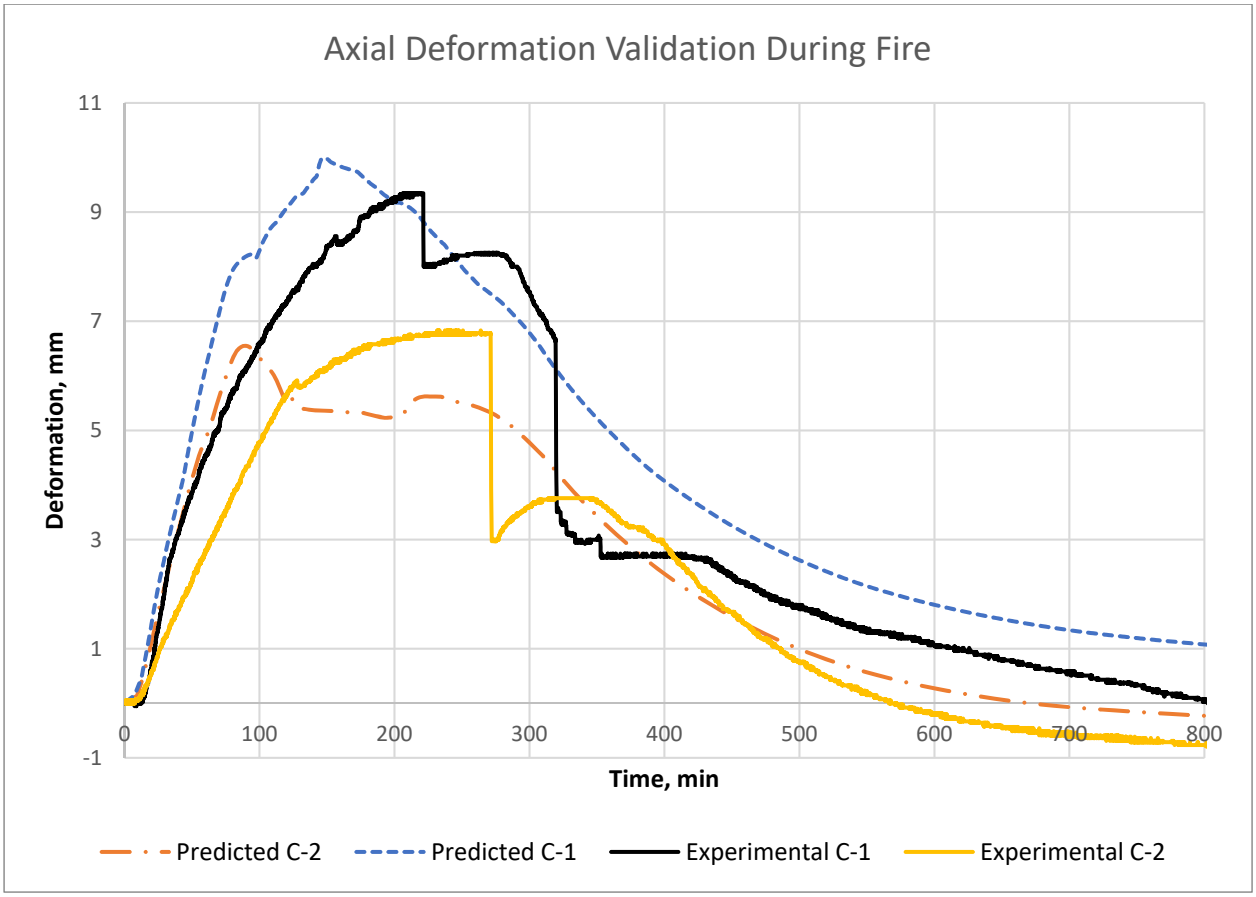

Figure 4.6. Axial Deformation Validation During Fire Exposure and Cooling.

4.5.3 Residual Response after Fire Exposure

The load-deformation response from the numerical model and experimental data for residual capacity are both plotted in Figure 4.7, for column C-1 and C-2. The predicted load-deformation response of column C-1 and C-2 depicts two distinct stages of response, i.e. linear deformation followed by plastic deformation as the applied load increases. The predicted residual response agrees well with the measured response, however, the predicted residual capacity for each column is higher by 14 and 18 percent for column C-1 and C-2, respectively. This can be attributed to corner peeling (spalling) seen in the experiment, but is not explicitly accounted for in the model.

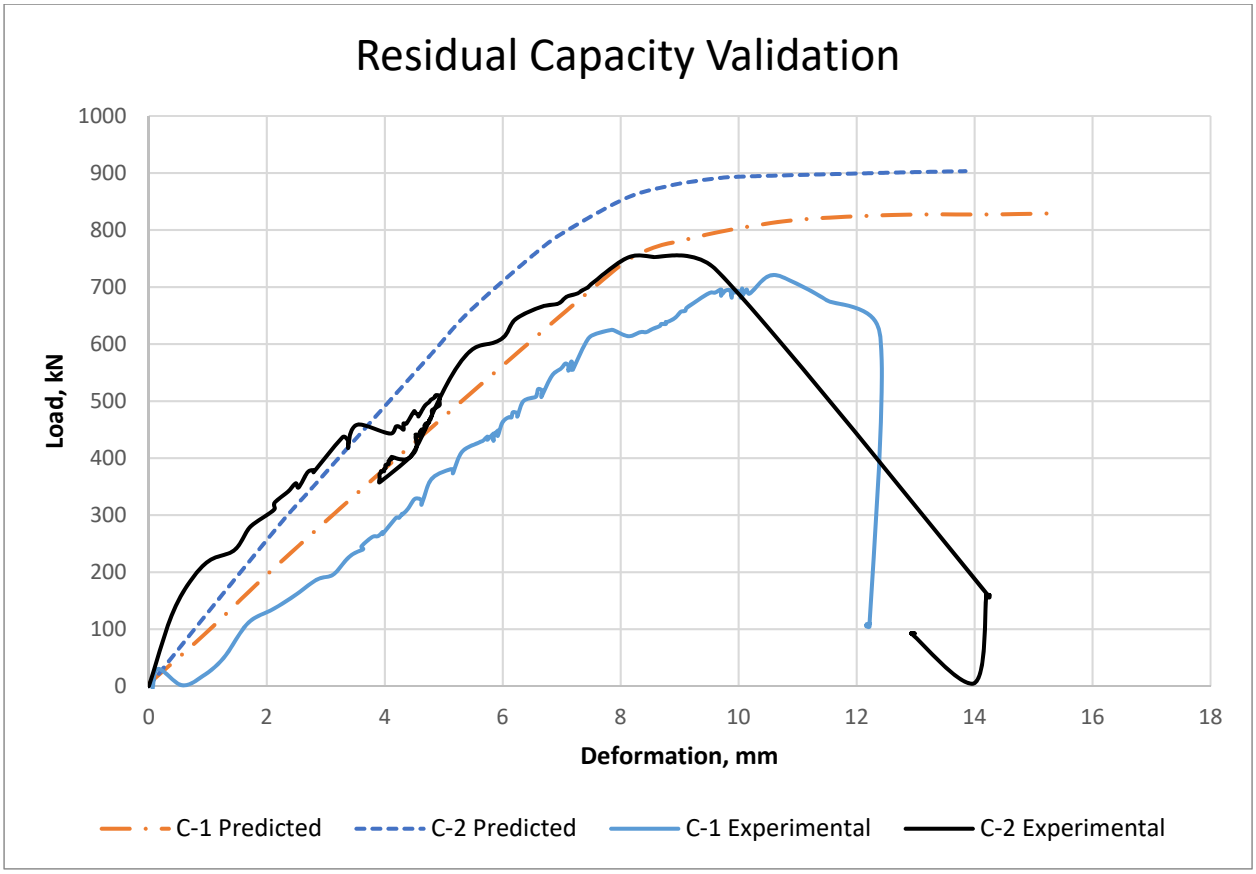

Figure 4.7. Load-Deformation Validation of Residual Capacity.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Fire tests on two RC columns were conducted under varying fire and load level conditions. The columns comprised of two NSC columns, one which was cast in January 2017 while the other was cast in approximately 2009. Each column was tested under two different design fire scenarios. Data from the fire tests was utilized to validate the numerical model for various response parameters such as temperatures, deformations and post fire capacity.

The fire resistance of RC columns is currently evaluated based on standard fire conditions with no consideration given to the cooling phase after the fire. The residual capacity of RC columns after fire exposure is also not immediately clear. This is primarily due to the lack of understanding on the response of RC columns under realistic fire, loading, and restraint conditions. To study the response of RC columns under realistic fire conditions, as well as their post fire capacity, a numerical model was developed for tracing the response of RC columns under realistic fire, loading and exposure conditions. The model is based on a finite element approach and temperature dependent material properties and temperature applied as an external load to trace the response of the columns from the pre-fire stage to failure (if present) under fire conditions. The model takes the residual stresses and strains after cooling of the fire and are applied as a load in the residual capacity stage of analysis. The critical factors, namely; high temperature material properties, different strain components, different exposure conditions and concentric loading, that have significant influence on the fire response of RC columns, are accounted for in the analysis.

The data collected from the residual capacity tests was used to validate the residual capacity stage of analysis from the numerical model. The validated model can reasonably predict the thermal and

structural response of a RC column from a fire exposure as well as said columns residual capacity after cooling.

5.2 Key Findings

Based on the information presented in this study, the following key conclusions are drawn:

- There is limited information on the fire performance of RC columns, especially under design fire scenarios. Also, to date, there is no rational approach in determining the residual capacity of RC columns after exposure to fire.
- Data from fire exposure and residual capacity tests on the two RC columns indicate that:
    - Peak reinforcing steel temperatures can occur up to 90 minutes and 80 minutes after the completion of heating for a 90-minute and 120-minute fire exposure, respectively. Due to this fact, to better characterize fire performance of RC members, a realistic fire scenario with a well-defined cooling phase is necessary.
    - The residual capacity of RC columns, similar to the ones tested in this study, can be up to 34% and 29% of its nominal (un-factored) capacity for a 90-minute and 120-minute fire exposure, respectively.
    - Use of UPV testing on a fire exposed RC column can provide practitioners an indication of the extent of thermal damage on said member. The observed decay in pulse velocity between the fire damaged concrete layer and undamaged concrete, can provide a reasonable estimation of the residual concrete compressive strength in the damaged layer. It should be noted that average pulse velocity values of concrete members not exposed to fire are required in applying this method for obtaining the compressive strength of the fire damaged layer of concrete.

- The proposed finite element model can predict the fire response of RC columns ranging from the capacity of the columns before fire exposure to their capacity after cooling down from fire exposure. The model accounts for critical parameters such as different fire scenarios, high temperature material properties, various strain components and restraint effects.
- While much of the ACI design capacity is retained in the columns tested in this study, the realistic nominal capacity, calculated by using actual compressive strength of concrete, shows a significant *drop* in residual capacity. This fact is ascribed to the variety of safety factors that are built into the design of RC columns, as well as the real compressive strength of concrete being nearly twice that of the specified (f'_c) value.
- There are a significant number of inter-dependent variables that will affect the post-fire capacity of an RC column. Namely, the duration of fire exposure will have a substantial impact on the magnitude of peak temperatures experienced as well as how long after the start of cooling those temperatures occur. This in turn will affect the residual mechanical properties of constituent materials.

5.3 Recommendations for Future Research

While this study has advanced the state-of-the-art with respect to the fire response of RC columns, and their residual capacity after fire, further research is required to fully characterize the complex behavior of concrete columns. The following are some of the key recommendations for further research in this area:

- The finite element model can be further advanced by incorporating fire induced spalling of concrete.

- Further UPV tests are needed at both the material and structural levels. This will give better understanding as to the sensitivity of UPV testing, specifically in determining the effect of various measurement distances as well as employing other pulse velocity measurement techniques. Moreover, much refinement is needed in determination of the depth of concrete damage in fire exposed concrete members.
- Additional residual fire tests are needed to study the influence of other parameters, such as the effect of different cross section sizes, reinforcement levels, various levels of confinement of concrete in the column center as well as various restraint conditions. Moreover, the effect that water quenching has on the residual capacity of fire exposed members should be explored.

5.4 Research Impact

The extent of damage as well as the residual capacity of a fire exposed RC column may not be immediately obvious in post-fire investigations. The research undertaken as part of this study is to help provide a fundamental understanding of the thermo-mechanical response of a RC column exposed to realistic fire scenarios. Furthermore, this study was intended to investigate the residual capacity of fire exposed RC columns, through numerical modeling and validating the model with experimental studies. The proposed numerical model offers a convenient way to estimate not only the room temperature capacity, but thermo-mechanical response during fire, and the residual response after fire exposure for RC columns. The model is also a cost-effective alternative to conducting full scale fire tests to characterize the fire resistance, as well as post-fire capacity, opposed to testing cores from fire damaged structures.

Further, the experimental results from this study indicates that the characterization of RC members with fire resistance experiments alone may not be adequate in capturing the full thermo-mechanical

response. Peak temperatures in the concrete cross sections that occur long after a realistic fire exposure has ceased are a clear indicator that further thermal damage to the member is possible after the heating phase of a fire. Moreover, the residual capacity retained by a fire exposed RC column is highly variable. It is a function of the severity of fire exposure, temperature dependent material properties of concrete and reinforcing steel during heating, cooling and residual stages, load (stress) level, restraint of the column during fire exposure and the post-fire residual deformations.

To date, there are no numerical studies that account for all key factors in residual capacity evaluation. There is also a lack of data on the residual response of NSC columns after exposure to a realistic fire, under load. Therefore, the developed numerical model, validated with the presented experimental results, will allow practitioners to account for the key factors that affect the residual capacity of a fire exposed RC column. Such an assessment is not possible using the currently available approaches, and the proposed numerical model will allow development of optimal retrofitting strategies after fire exposure.

APPENDICES

APPENDIX A: High Temperature Material Relationships

A.1 Constitutive Relations for Concrete

Table A.1. Stress-Strain for Concrete at Elevated Temperatures, adopted from EC4 [10].

Strain Range	Stress, $\sigma_{c,\theta}$
$0 \leq \varepsilon_{c,\theta} < \varepsilon_{cu,\theta}$	$\sigma_{c,\theta} = \frac{3\left(\frac{\varepsilon_{c,\theta}}{\varepsilon_{cu,\theta}}\right)}{\left\{2 + \left(\frac{\varepsilon_{c,\theta}}{\varepsilon_{cu,\theta}}\right)^3\right\}}$
$\varepsilon_{c,\theta} \geq \varepsilon_{cu,\theta}$	Linear descending branch is adopted.
Where: $\varepsilon_{c,\theta}$=Temperature dependent strain $\varepsilon_{cu,\theta}$=Ultimate temperature dependent strain $\sigma_{c,\theta}$=Temperature dependent stress level at a given strain Note: The values for $\varepsilon_{c,\theta}$ are determined from Table A.2.	

Table A.2. Temperature Dependent Concrete Strengths and Associated Strains, adopted from EC2 [20].

Concrete Temperature, θ (°C)	Calcareous Aggregates		
	$f'_{c,\theta}/f'_c$	$\varepsilon_{c,\theta}$	$\varepsilon_{cu,\theta}$
20	1.00	0.0025	0.0200
100	1.00	0.0040	0.0225
200	0.97	0.0055	0.0250
300	0.91	0.0070	0.0275
400	0.85	0.0100	0.0300
500	0.74	0.0150	0.0325
600	0.60	0.0250	0.0350
700	0.43	0.0250	0.0375
800	0.27	0.0250	0.0400
900	0.15	0.0250	0.0425
1000	0.06	0.0250	0.0450
1100	0.02	0.0250	0.0475
1200	0.00	-	-

Table A.3. Specific Heat of Concrete, adapted from EC2 [20].

Temperature Range, θ (°C)	Specific Heat, $c_p(\theta)$ (J/kgK)
$20 \leq \theta \leq 100$	900
$100 < \theta \leq 200$	$900 + (\theta - 100)$
$200 < \theta \leq 400$	$900 + \frac{(\theta - 200)}{2}$
$400 < \theta \leq 1200$	1100

Table A.4. Thermal Conductivity of Concrete, adapted from EC2 [20].

Concrete Temperature, θ (°C)	Thermal Conductivity, λ (W/mK)
$20 \leq \theta \leq 1200$	$\lambda = 1.36 - 0.136\left(\frac{\theta}{100}\right) + 0.0057\left(\frac{\theta}{100}\right)^2$

A.2 Constitutive Relations for Reinforcing Steel

Table A.5. Stress-Strain Relation for Reinfrocing Steel, adapted from [20].

Steel Strain, ε_s	Steel Stress, σ_s
$\varepsilon_s \leq \varepsilon_{sp,T}$	$\sigma_s = \varepsilon_s E_{s,T}$
$\varepsilon_{sp,T} < \varepsilon_s \leq \varepsilon_{sy,T}$	$\sigma_s = f_{sp,T} - c + \left(\frac{b}{a}\right)\left(a^2 - \left(\varepsilon_{sy,T} - \varepsilon_s\right)^2\right)^{0.5}$
$\varepsilon_{sy,T} < \varepsilon_s \leq \varepsilon_{st,T}$	$\sigma_s = f_{sy,T}$
$\varepsilon_{st,T} < \varepsilon_s \leq \varepsilon_{su,T}$	$\sigma_s = f_{sy,T}\left(1 - \frac{\varepsilon_s - \varepsilon_{st,T}}{\varepsilon_{su,T} - \varepsilon_{st,T}}\right)$
$\varepsilon_s > \varepsilon_{su,T}$	0.0

Where:
ε_s = Current strain level in steel
$\varepsilon_{sp,T} = \frac{f_{sp}}{E_{s,T}}$, temperature dependent strain at the proportionality limit
$\varepsilon_{sy,T}$ = Yield strain, 0.02
$\varepsilon_{st,T}$ = Strain at tensile strength, 0.15
$\varepsilon_{su,T}$ = Strain at rupture, 0.2
f_{sp} = Stress at proportionality limit
$f_{sy,T}$ = Yield stress
$E_{s,T}$ = Young's Modulus of Steel; dependent on temperature

$$a^2 = \left(\varepsilon_{sy,T} - \varepsilon_{sp,T}\right)\left(\varepsilon_{sy,T} - \varepsilon_{sp,T} + \frac{c}{E_{s,T}}\right)$$

$$b^2 = c\left(\varepsilon_{sy,T} - \varepsilon_{sp,T}\right)E_{s,T} + c^2$$

$$c = \frac{\left(f_{sy,T} - f_{sp,T}\right)^2}{\left(\varepsilon_{sy,T} - \varepsilon_{sp,T}\right)E_{s,T} - \left(f_{sy,T} - f_{sp,T}\right)}$$

Values of $f_{sp,T}$, $f_{sy,T}$ and $E_{s,T}$ can be obtained in Table A.6

Table A.6. Temperature Dependent Stress Values for Reinforcing Steel, adapted from [20].

Steel Temperature, T (ºC)	f_{yT}/f_y	f_{sp}/f_y	E_{st}/E_s
20	1.00	1.0000	1.0000
100	1.00	1.0000	1.0000
200	1.00	0.8070	0.9000
300	1.00	0.6130	0.8000
400	1.00	0.4200	0.7000
500	0.78	0.3600	0.6000
600	0.47	0.1800	0.3100
700	0.23	0.0750	0.1300
800	0.11	0.0500	0.0900
900	0.06	0.0375	0.0675
1000	0.04	0.0250	0.0450
1100	0.02	0.0125	0.0225
1200	0.00	0.0000	0.0000
Where: f_y and E_s are yield stress and Young's Modulus at room temperature, respectively			

Table A.7. Specific Heat of Reinforcing Steel, adapted from [10].

Steel Temperature Range, T (ºC)	Specific Heat of Steel, C_s (J/kgK)
$20 \leq T \leq 600$	$425 + 0.773T - 0.00169T^2 + 2.22x10^{-6}T^3$
$600 < T \leq 735$	$666 - \left(\frac{13002}{T-738}\right)$
$735 < T \leq 900$	$545 + \left(\frac{17820}{T-731}\right)$
$900 < T \leq 1200$	650

Table A.8. Thermal Conductivity of Reinforcing Steel, adapted from [10].

Steel Temperature, T (°C)	Thermal Conductivity, λ (W/mK)
$20 \leq T \leq 800$	$\lambda = 54 - 3.33x10^{-2}T$
$800 < T \leq 1200$	27.3

APPENDIX B : Load Carrying Capacity Calculation for Tested Columns

The following procedure has been adopted from Raut (2011) [48], as well as following provisions from ACI 318 [2].

Table B.1. Assumed Parameters for Column Design.

Parameter	Value
Section Dimensions	203x203 mm
Longitudinal Steel Reinforcement	4x 19mm (#6) bars
Yield Strength of Steel	414 MPa
Column Height	3350 mm
Concrete Cover	38.1 mm

LOAD CALCULATIONS

Length of column = 3350 mm

Assumed pin ended restraint conditions.

Radius of gyration $r = 0.3h$, where h is the dimension of the column in the direction of eccentricity

$h = 203$ mm

$r = 0.3 \times 203 = 60.9$ mm

The Slenderness ratio is computed as:

$$\frac{KL_u}{r}$$

Where:

K=effective length factor, 1.0 for pinned ends in this case

L_u = unbraced length of column

r = radius of gyration

Slenderness ratio $= \frac{KL_u}{r} = \frac{(1.0)(3350)}{60.9} = 55 > 22$ which implies that the column is slender and the slender and moment magnification procedure should be used per ACI 318 [2].

Load Carrying Capacity Assuming Column to be Slender

Modulus of Elasticity of concrete

$E_c = 0.043 w_c^{1.5} \sqrt{f'_c} = 0.043(2402.77)^{1.5}\sqrt{27.58} = 26{,}597$ MPa

Gross moment of inertia

$I_g = \frac{bh^3}{12} = \frac{203*203^3}{12} = 141.5 \times 10^6 \text{mm}^4$

Assuming stiffness reduction factor accounting for sustained axial loads to be $\beta_d = 0.6$

Flexural Rigidity:

$EI = \frac{E_c \frac{I_g}{2.5}}{1+\beta_d} = \frac{(26597)\frac{141.5\times10^6}{2.5}}{1+0.6} = 9.4087 \times 10^{11}$ Nmm^2

Euler Buckling Load:

$P_{cr} = \frac{\pi^2 E I_g}{(KL_u)^2} = \frac{\pi^2 \times 26597 \times 141.5 \times 10^6}{(1.0\times3350)^2} = 3309.8$ kN

Moment Magnification Factor:

$$\delta = \frac{C_m}{1 - \frac{P_u}{0.75P_{cr}}} \geq 1.0 = \frac{1.0}{1 - \frac{P_u}{0.75P_{cr}}}$$

Actual moment carrying capacity:

$$M_n = \delta P_u e_{min}$$

And the minimum eccentricity, e_{min}, is calculated as:

$e_{min} = 0.6 + 0.03 \times h = 0.6 + 0.03 \times 8inch = 0.84\ inch = 21.34$ mm

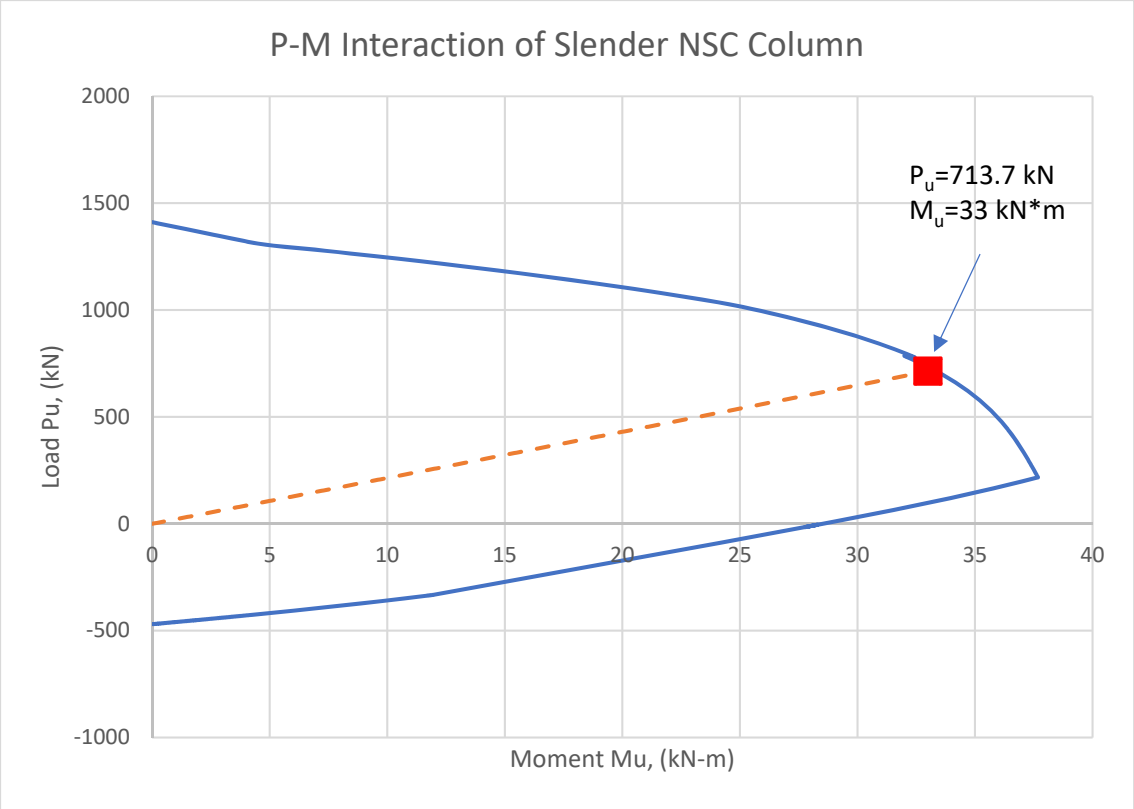

Figure B.1. Moment-Curvature Interaction with Max Values of P_u and M_u noted.

$P_u = 713.7$ kN

$M_u = 33$ kN.m

$e = 21.34$ mm

Table B.2. Input Variable for Capacity Calculations.

Parameter	Value
Strength Reduction Factor, ϕ	0.65
Gross Section Area, A_g (mm^2)	41,209
Area of Longitudinal Reinforcing Steel, A_s (mm^2)	1134.12
28-day concrete compressive strength, f'c (MPa)	27.58
Yield strength of reinforcing steel, f_y (MPa)	414

The nominal axial capacity of a RC column is computed as:

$P_0 = 0.85f'_c(A_g - A_s) + f_y A_s = 0.85(27.58)(41209 - 1134.12) + (414)(1134.12) =$

1409 kN

The factored maximum axial capacity as per ACI 318 [2] is computes as:

$\emptyset P_{0,max} = 0.8\emptyset P_0 = (0.8)(0.65)(1409) =$ **732 kN**

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