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PERFORMANCE IN FIRE OF FRP-CONFINED REINFORCED CONCRETE COLUMNS

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ABSTRACT: Over the past decade, research has shown that fibre reinforced polymers (FRPs) can be efficiently, economically, and safely be used for strengthening and rehabilitation of reinforced concrete (RC) structures. However, relatively little is known about the behaviour of FRP materials at high temperature, and this is a primary factor limiting the widespread application FRP materials in buildings, parking garages, and industrial structures. This paper presents the results of a numerical and experimental program to investigate the fire performance of FRP-wrapped (confined) RC columns. The primary objectives of this research project were: to experimentally investigate the behaviour in fire of FRP-wrapped and insulated RC columns; to develop numerical models to simulate the behaviour in fire of these members; to investigate techniques to improve their behaviour in fire; and to use data from experimental and numerical studies to provide fire-design guidance. Experimental data are presented from full-scale fire endurance tests conducted on three FRP-wrapped RC columns at the National Research Council of Canada (NRC). A numerical model is presented, which is capable of predicting the thermal and structural response of an FRP-wrapped concrete column under exposure to a standard fire. The model is shown to adequately predict the observed response of FRP-wrapped columns in fire. It is demonstrated that, while currently available FRPs are sensitive to the effects of elevated temperatures, appropriately designed and insulated FRP-wrapped RC columns are capable of achieving satisfactory fire endurances.

1. INTRODUCTION

Externally-bonded fibre reinforced polymers (FRPs) are now widely accepted as an effective and efficient means of repairing and upgrading deteriorated or under-strength reinforced concrete (RC) structures. One of the most efficient, and widely implemented, of these FRP repair techniques is circumferential wrapping (confinement) of RC columns, which has been shown to increase both the axial strength and ductility of these members. Design recommendations are now available for repair and upgrade of concrete columns with FRP wraps, and the technique has, in the last 10 years, been used in hundreds of field applications around the world. Despite the numerous advantages of the FRP wrapping technique, it has yet to see widespread application in buildings, due in large part to uncertainties associated with the performance of FRP repair materials and FRP repaired concrete members during fire. Most building structures are subject to strict building code requirements for flame spread, smoke generation, and maintenance of structural integrity during fire, and there is currently insufficient information on FRP strengthening systems under these conditions. Indeed, several studies [1] have recently placed fire among the most critical research needs to promote further application of FRPs in structural applications. This paper presents results of an ongoing experimental and numerical research program investigating the behaviour in fire of FRP-wrapped RC columns. The current discussion focuses on column strengthening applications, although slab and beam-slab assembly strengthening applications are also being investigated within the overall program.

2. BACKGROUND

2.1 Fire Endurance

In considering the fire performance of FRP-wrapped RC columns, it is important to first outline what is implied by "fire endurance". The fire endurance (fire resistance) of structural assemblies is currently defined in North America by ASTM E119 [2] or CAN/ULC S101 [3]. For columns, the only structural requirement to achieve satisfactory fire endurance is that they must be able to carry their full service load for the required duration during fire. The required duration (fire rating) is generally between 2 and 4 hours and depends on a variety of factors including the type of building material and the structure's use and occupancy. Under current fire testing and design guidelines there is no explicit requirement that the FRP temperature remain below some specified value (e.g. the matrix glass transition temperature (GTT)).

It is worth noting that neither ASTM E119 nor CAN/ULC S101 explicitly address the important issues of flame spread or smoke generation and toxicity, but that these factors are potentially critically important in the design of structural systems for buildings. This is particularly true for externally-bonded FRP systems, since they are generally combustible, unlike steel and concrete. Flame spread and smoke generation considerations for FRP materials are omitted from the current discussion, and interested readers are encouraged to consult specialized references [4, 5] or other ASTM or CAN/ULC standards for more information on this topic.

2.2 FRPs in Fire

All structural materials, including concrete and steel, experience some degradation in mechanical properties at elevated temperature, and this is true also of FRPs. At elevated temperatures, beyond the GTT of the FRP's polymer matrix component, mechanical properties deteriorate rapidly. The resulting loss of load transfer between the fibres, in conjunction with a less severe deterioration in the mechanical properties of the fibres themselves, results in a reduction in the strength and stiffness of the FRP. In addition, in externally-bonded FRP applications, it is likely that exposure to elevated temperatures would lead to rapid and severe deterioration of the FRP/concrete bond, resulting in delamination of the FRP and loss of its effectiveness as tensile or confining reinforcement.

Data available in the literature, obtained from tensile tests on a variety of FRP materials at elevated temperature, have confirmed the sensitivity of FRP materials to extreme temperatures. However, the data cover a range of fibre and matrix types, and it is thus difficult to make generalizations regarding the observed deterioration of mechanical properties for various types of FRPs. For the purposes of the numerical models presented later, it was required to approximate the high-temperature mechanical behaviour of FRP materials such that this information could be used to predict the strength of FRP-confined concrete columns during fire [6]. To accomplish this goal, a database of results from tensile tests on FRP at high temperature was assembled from the literature. For each type of FRP (i.e. glass/epoxy, carbon/epoxy, and aramid/epoxy), a sigmoid function was fitted to the data using a least-squares regression analysis [6]. A similar approach has been taken by Katz and Berman [7] to describe deterioration of the bond between FRP rebars and concrete at high temperature. As an example, the resulting curves for the strength and stiffness deterioration of carbon/epoxy FRP (CFRP) with temperature are shown in Figure 1. Also included in Figure 1 are equivalent curves for reinforcing steel and concrete as given by Lie [8]. The sensitivity of FRP materials to elevated temperatures is clearly evident. It is important to recognize that the curves in Figure 1 do not consider the potential loss of bond at high temperature. Bond tests on FRPs at high temperature are required to obtain information in this regard.

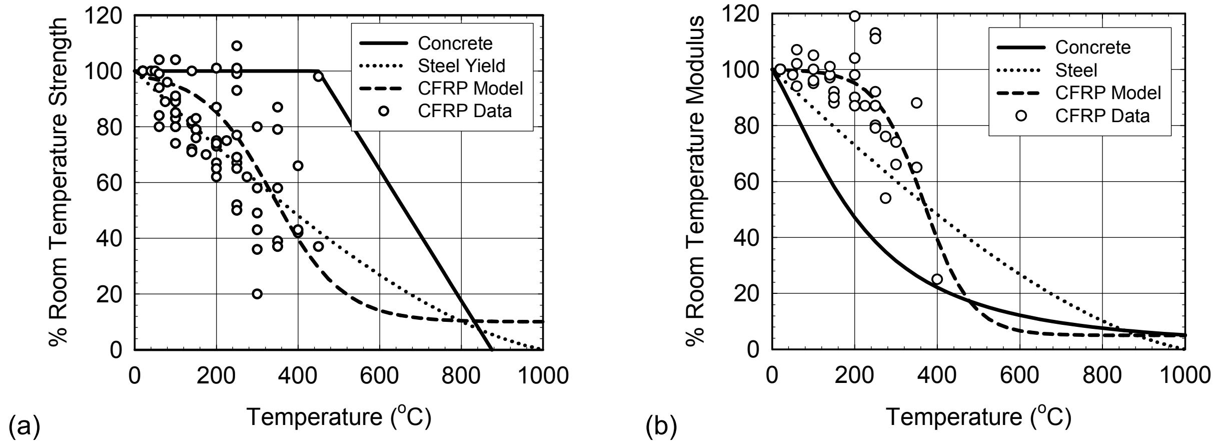

Fig. 1 – **Reduction of (a) strength and (b) stiffness of carbon/epoxy FRP at elevated temperature**

The potentially harmful effects of fire and high-temperature on FRP-strengthened concrete members are well recognized in the literature, although relatively few studies have been conducted investigating this issue. Fire tests on FRP-plated RC beams and/or slabs have been conducted by Deuring [9] and Blontrock et al. [10], and limited work on the residual performance of FRP-wrapped cylinders after exposure to elevated temperatures has been reported by Saafi and Romine [11]. All of these studies have demonstrated the sensitivity of FRP materials to elevated temperature and have confirmed the need for thermal insulation of FRP materials during fire to prevent rapid loss of the FRP's structural effectiveness. However, no information is currently available on the performance in fire of FRP-wrapped RC columns, and it is this knowledge gap that the current paper addresses.

Although never reported in the literature, anecdotal information on the fire performance of FRP-wrapped concrete columns exists from full-scale fire tests performed in the late 1990's. Three tests, performed on full-scale GFRP-wrapped columns demonstrated that unprotected FRP wraps will be lost very rapidly (within the first 30 minutes) during fire exposure, and that supplemental insulation, applied to the exterior of the FRP, is required to maintain effectiveness of the wraps during fire. As a result, the current study considers only FRP-wrapped *and insulated* columns; uninsulated wraps have not been tested.

3. EXPERIMENTAL PROGRAM

The experimental program consisted of full-scale fire endurance tests on FRP-wrapped RC columns, with supplemental insulation applied to the exterior of the FRP wrap. Two circular columns and one square column have been tested to date. Ancillary tests on constituent materials have also been performed.

3.1 Specimen Details and FRP Wrapping/Insulation

Specimen dimensions, reinforcement details, and material properties for all three columns are presented in Table 1. All were cast using Type 10 cement concrete and incorporated Grade 400 internal reinforcing steel. FRP wrap and insulation details for all three columns are presented in Table 2. Both circular columns were wrapped in the circumferential direction with a single layer of the Tyfo® SCH carbon FRP strengthening system. The square column, which was intended to be representative of an actual potential field application, was wrapped with three circumferential layers of Tyfo® SEH glass FRP system.

All three columns were provided with a unique supplemental insulation system. This system, called Tyfo® VG/EI, is a two-part insulation system developed specifically for fire protection of externally-bonded FRPs. The system consists of a low density, low thermal conductivity, spray-applied cementitious fire insulation (VG), which is sealed with a surface coat of a modified epoxy surface hardening and sealing compound (EI). Insulation thicknesses were varied among the columns in an attempt to obtain information on the amount of insulation required to maintain certain allowable temperatures in the FRP wraps.

Table 1 – Specimen dimensions, reinforcement details, and material properties.

No.	Dimensions	Height (mm)	Primary Reinforcement	Hoop Reinforcement	Agg. Type*	f'_c (MPa)
1	400 mm Ø	3810	8 – 20 mm Ø bars	10 mm spiral – 50 mm pitch	CA	40
2	400 mm Ø	3810	8 – 20 mm Ø bars	10 mm spiral – 50 mm pitch	CA	39
3	400 x 400 mm	3810	4 – 25mm Ø bars	10 mm ties – 400 mm spacing	SA	52

* CA – carbonate aggregate, SA – siliceous aggregate

Table 2 – FRP wrap and insulation details used in the experimental program.

No.	FRP Type*	# Layers	VG thickness (mm)	EI Thickness (mm)	Fire Test Load Ratio**
1	Tyfo® SCH Carbon / Tyfo® S Epoxy	1	32	0.56	0.5
2	Tyfo® SCH Carbon / Tyfo® S Epoxy	1	57	0.25	0.5
3	Tyfo® SEH Glass / Tyfo® S Epoxy	3	38	0.25	0.69

* for additional information on these FRP systems consult www.fyfeco.com.
** ratio of load applied during the fire test to the design ultimate load of the strengthened column (with design ultimate load calculated according to ACI 440.2R-02 [14]).

3.2 Fire Test Procedures

The columns were tested individually by exposing them to a standard fire [3, 4] while under load in the column test furnace at NRC. The sustained load applied during the tests was calculated according to the requirements of current fire testing guidelines [3, 4], by back-calculating the service load from the ultimate factored load assuming a dead-to-live load ratio of 1:1. For the circular columns, the ultimate load was determined using the guidelines set out by ISIS Canada Design Manual No. 4 [12], while for the square column this load was determined using the recommendations of Teng et al. [13] (to determine the FRP-confined concrete strength) in conjunction with the recommendations of ACI 440.2R-02 [14].

The three columns were extensively instrumented with thermocouples and were monitored throughout the tests. In addition, applied load and overall axial elongation of the columns were measured and recorded during testing. Fire exposure was continued until structural failure of the columns occurred.

4. NUMERICAL MODEL

Fire resistance experiments on loaded full-scale columns are relatively complex and costly to perform. With the advent of powerful computing capabilities, however, it is now possible to develop detailed numerical models which, once verified by relatively few full-scale experiments, can be used to conduct parametric studies and examine the influence of various factors on member behaviour in a more cost-effective manner. For the current study, a numerical model has been developed to simulate both the heat transfer and load capacity of FRP-wrapped RC columns. At present, the model can treat only circular columns. An extension of the model to treat square and rectangular columns is currently under development. A more complete discussion of the numerical model, which is similar in methodology to models developed by Lie [8], is provided in [15].

4.1 Heat Transfer

The transfer of heat within the column during fire is treated using an explicit finite difference methodology wherein the column is divided into a series of ring elements. For each ring, an energy balance is formulated such that the heat entering into minus the heat going out of the element (due to conduction) is equated to the energy stored in the element during some finite time interval. This allows the development of a series of equations which, once programmed into a computer, can be used to predict the temperature at any location within an FRP-wrapped and insulated RC column during exposure to fire following a known time-temperature curve.

The non-linear variation in the thermal properties of the insulation, FRP, and concrete are accounted for in the analysis using a series of thermal subroutines which update material properties within a ring element based on its temperature during the previous time step. The thermal analysis ignores the contribution of the reinforcing steel to temperature propagation since its effect is negligible due to the small cross-sectional area of the rebars [8]. The boundary condition at the fire/column interface is treated by assuming that heat is transferred due to radiation only, an assumption which has been successfully used in a variety of previous studies performed at NRC and elsewhere [8].

4.2 Load Capacity

The load capacity of a structural member in fire can be evaluated once the distribution of internal temperatures is known (using the heat transfer model described above). The analysis tools developed in the current program allow for the calculation of column strength based on pure axial crushing or on buckling. It is worth noting that buckling generally governs failure in fire for column lengths that would be encountered in practice. The analysis accounts for the thermal deterioration in mechanical properties of all materials involved, except for the insulation which is assumed to provide negligible strength to the column.

The thermal deterioration in mechanical properties of steel and concrete is treated in the model using mathematical relationships suggested by Lie [8] and shown in Fig. 1, whereas the deterioration of FRP is assumed to follow the previously discussed sigmoid functions, also shown in Fig. 1. The beneficial effect of FRP confinement on the stress-strain behaviour of the concrete in the column is treated using a Spolestra and Monti [16] iterative confinement routine, which has been modified to account for the deterioration in mechanical properties of both concrete and FRP with temperature and for differential thermal expansion between the concrete and the confining FRP wrap. The output of the load capacity analysis consists of curves showing the variation in axial strength, buckling strength, or overall axial elongation, with time during exposure to fire.

5. RESULTS AND DISCUSSION

5.1 Temperatures

Figure 2 presents temperatures recorded at various key locations within Cols. 2 & 3 during testing. Included also in Figure 2a are equivalent temperatures as predicted by the numerical model. For both columns, the temperature at the level of the FRP is seen to increase fairly rapidly within the first 15-45 minutes of exposure, at which point the rate of temperature rise decreases and a temperature plateau is seen near 100˚C. The duration of this plateau, which can be attributed to the evaporation of both free and chemically-combined moisture from the insulation at temperatures near the boiling point of water, is longer for Col. 2, which has a greater insulation thickness, as should be expected. Indeed, the FRP temperature in Col. 2 remains less than 100˚C for more than three hours under fire exposure. Once all of the moisture has evaporated, temperatures at the level of the FRP increase more rapidly until the end of the test. This behaviour implies that one way to significantly improve the fire performance of the columns (as insulated herein) would be to increase the GTT of the polymer matrix to even slightly above 100˚C. However, as will be demonstrated below, keeping the FRP temperature below the GTT is not a necessary criterion for adequate fire endurance. The temperature at the level of the FRP remained less than the matrix ignition temperature for the full duration of fire exposure for Col. 2. For Col. 3, the ignition temperature was exceeded at about 3 hours of exposure (a factor which may have contributed to its sudden failure at slightly more than 4 hours).

For all three columns, the thermal protection provided by the supplemental insulation was excellent, and temperatures within the concrete and reinforcing steel remained less than 350˚C for the full duration of the fire (until failure). Thus, it is likely that the columns retained essentially all of their unwrapped (pre-existing) strength until the insulation was lost late in the fire exposure (beyond 4 hours for Col. 5 and beyond 5 hours for Cols. 1 & 2). Hence, the columns satisfied the ASTM E119 (or CAN/ULC S101) fire endurance requirement for 4 or 5 hours for the square and circular columns respectively.

Figures 2a and 2b show that, while the predictions of the numerical model generally is in agreement with test data, the model does not precisely capture the 100˚C temperature plateau exhibited in the experimental thermal profiles. This can be attributed to the fact that, while the model does account for the evaporation of moisture from individual elements at 100˚C, it does not account for the migration of free moisture in the concrete away from the fire [8]. However, the model satisfactorily predicts the temperatures at key locations in the column for the purposes of approximating the columns' load capacity.

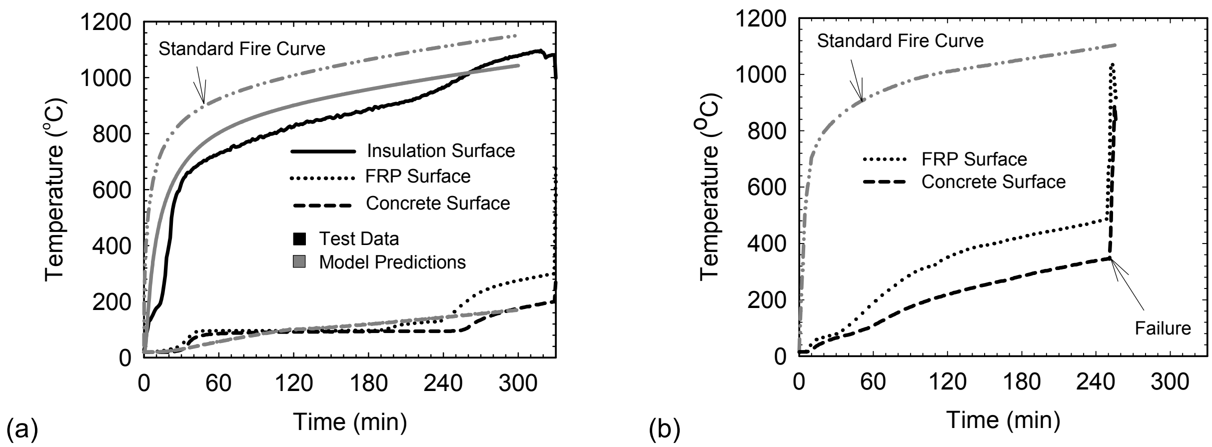

Fig. 2 – **Temperatures observed (or predicted) in (a) Col. 2 (57 mm VG) and (b) Col. 3 (38 mm VG).**

5.2 Fire Endurance

All columns tested were able to carry their full service load for at least four hours of exposure to the standard fire. Cols. 1 and 2 both failed at approximately 5.5 hours of fire exposure, and only once the applied load had been increased to about 1.8 times the required service load. Failure was sudden and explosive, and was accompanied by extensive spalling. Thus, as expected given the thermal profiles of concrete discussed above, these two columns indeed retained virtually all of their room-temperature unwrapped strength for the full duration of fire exposure. Col. 3 (the square column) failed in an explosive manner at about 4.25 hours of fire exposure while under only its applied service load. The failure of Col. 3 was likely initiated moments before it actually failed, when minor localized spalling caused a major loss of fire insulation. The resulting combustion of the fire-exposed FRP directly exposed the substrate concrete, which had previously been at less than 400˚C, to the full heat of the fire (at about 1100˚C). This resulted in an extreme thermal shock to the concrete and caused explosive spalling resulting in rapid and catastrophic failure of the column. Nonetheless, Col. 3 achieved a fire resistance rating in excess of four hours according to the ASTM E119 or CAN/ULC S101 guidelines.

To demonstrate the use of the numerical model for the prediction of fire endurance of an FRP-wrapped RC column, Fig. 3a shows the predicted axial crushing strength of a concrete column with fire exposure time, and Fig. 3b shows a similar plot for the column's predicted buckling strength. These plots were developed for circular columns with the same characteristics as those tested in the experimental program described herein (refer to Table 1). Also include in the plots are data points showing the failure points recorded in the two full-scale fire tests on circular columns conducted to date.

The following points are worthy of note with respect to Figure 3:

- The model reasonably predicts the strength of the two circular FRP-wrapped RC columns tested to date after 5.5 hours of fire exposure (based on the buckling analysis).
- For all cases shown the predicted axial crushing strength is greater than the predicted buckling strength for the full duration of the fire exposure (note that the buckling analysis assumes an initial eccentricity of 27 mm, as required by Clause 10.15.3 of CSA A23.3 [17]).

- Loss of structural effectiveness of the FRP is predicted to occur very rapidly during the fire exposure for a wrapped but uninsulated column. Once the wrap is lost, the strength of the column is only slightly greater than that of an equivalent unstrengthened column. Loss of the wrap is seen to be more significant for the crushing strength analysis as opposed to the buckling strength analysis. This is because confinement of concrete with an FRP wrap cannot be expected to significantly increase the concrete's modulus, and hence the buckling strength is not substantially improved by FRP wrapping.
- Even a small amount of supplemental insulation (32 mm for Col. 1) is predicted to significantly improve the retention of strength during fire. This is due primarily to the fact that the concrete and reinforcing steel in the column remain at sufficiently low temperatures to prevent degradation of their mechanical properties, and not to the effectiveness of the FRP wrap being maintained. Thus, the columns are predicted to retain a significant portion of their unwrapped strength even if the FRP is lost.

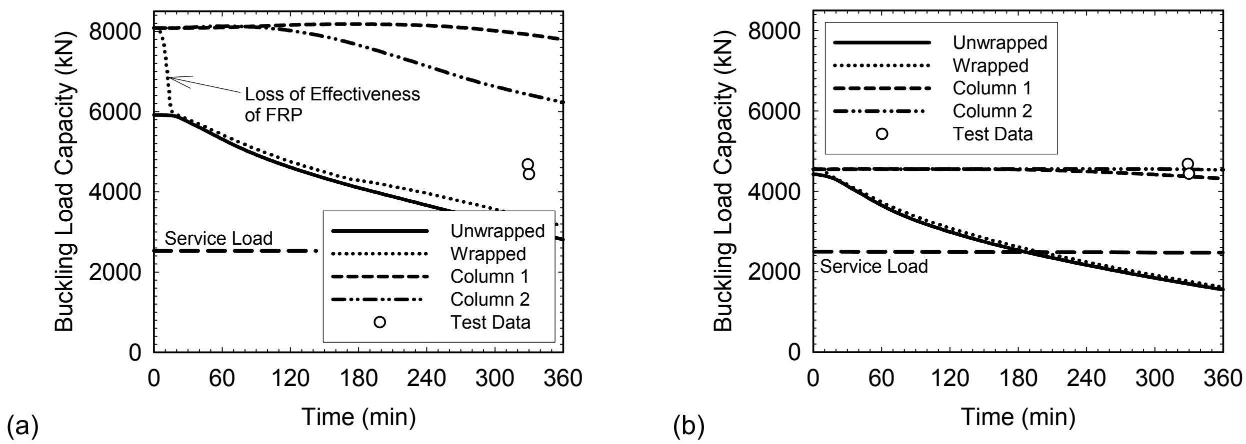

Fig. 3 – **Observed and predicted structural fire endurance for various column configurations based on (a) crushing strength and (b) buckling strength.**

6. CONCLUSIONS

Based on the FRP-wrapped RC column fire endurance studies conducted to date, the following conclusions can be drawn:

- FRP materials used as externally-bonded reinforcement for concrete structures are sensitive to the effects of elevated temperatures. FRPs experience degradation in strength, stiffness, and bond at temperatures exceeding the GTT of the polymer matrix.
- Appropriately designed (and in most cases supplementally-insulated) FRP-wrapped circular RC columns can achieve satisfactory fire endurances in excess of 5 hours based on the requirements of ASTM E119 or CAN/ULC S101.
- The numerical models presented briefly herein can be used to predict the heat transfer within, and load capacity of, unwrapped and FRP-wrapped and insulated RC columns under exposure to a standard fire. Parametric studies conducted using the model (not discussed herein) indicate that the most important factors influencing the fire endurance of FRP-wrapped RC columns are the thickness and thermal conductivity of the insulation applied to the exterior of the FRP wrap.
- While no explicit requirement currently exists that the temperature of an FRP wrap must remain below its matrix GTT during fire, it is not known what temperatures are allowable in the FRP such that it retains sufficient residual properties to remain effective after a severe building fire. Further work is required in this area.

7. CONSEQUENCES FOR DESIGN

The conclusions stated above support the following two fire-safety design recommendations:

(a) To protect against sudden and complete loss of effectiveness of FRP wraps during fire, the strengthened (FRP-wrapped) service load on the column should not exceed the design strength of the unstrengthened (pre-existing) column. This requirement is similar to a strengthening limit requirement currently suggested by ACI Committee 440 [14], and provides a measure of protection against poor installation practices or vandalism in addition to fire.

(b) Because fire design is concerned primarily with life-safety objectives, during fire it is essential that structural collapse is prevented. Thus, the nominal strength of the unstrengthened (pre-existing) column should remain greater than the service (unfactored) loads on the column for the required duration during fire. This is essentially a statement of the ASTM E119 or CAN/ULC S101 fire endurance criterion and confirms that loss of the effectiveness of the FRP is not explicitly a concern. Of course under this philosophy the increases in strength due to FRP wrapping must be limited to between 40% and 70% for columns designed using U.S. load factors and between 25% and 50% for columns designed using Canadian load factors (depending on the dead-to-live load ratio for the specific member being strengthened). This approach has also been taken by ACI Committee 440 [14]. Currently, the nominal strength of an FRP-wrapped and insulated RC column during fire can only be approximated using the numerical model presented briefly in this paper.

8. ACKNOWLEDGEMENTS

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