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Publisher's version / Version de l'éditeur:
https://doi.org/10.4224/20378290
Research Report (National Research Council of Canada. Institute for Research in Construction); no. RR-185, 2005-03-01

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Research Report No. 185

March 2005

Fire Research Program
Institute for Research in Construction
National Research Council Canada

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ABSTRACT

Three full-scale fire resistance tests were conducted at the National Research Council of Canada (NRC) in partnership with ISIS Canada and Fyfe Co. LLC, to investigate the behaviour of fibre reinforced polymer (FRP) wrapped (confined) reinforced concrete columns under exposure to the standard fire. Both columns were provided with a unique supplemental fire protection system. The results of fire endurance experiments on the three full-scale columns are described in this report. These tests were conducted as part of a collaborative research program, aimed at studying fire endurance of FRP-wrapped reinforced concrete columns. The test results suggest that, by providing proper fire insulation as described in this report, a 4-hour fire endurance rating can be achieved for loaded reinforced concrete columns strengthened with FRP wraps.

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INTRODUCTION

In recent years, the construction industry has shown significant interest in the use of fibre reinforced polymer (FRP) materials for reinforcement and strengthening of concrete structures. This interest can be attributed to the numerous advantages that FRP materials offer over conventional materials such as concrete and steel. One particularly successful use of FRPs in structural engineering applications involves repair and rehabilitation of existing reinforced concrete structures by bonding carbon/epoxy or glass/epoxy FRP strengthening systems to the exterior of reinforced concrete members.

In buildings, FRP strengthened structural systems members must be designed to satisfy the requirements of serviceability and safety limit states. One of the major safety requirements in building design is the provision of appropriate fire safety measures for structural members [1, 2]. The basis for this requirement can be attributed to the fact that, when other measures for containing the fire fail, structural integrity is the last line of defence for building occupants.

With the increased use of FRPs in structural applications, concern has developed regarding their behaviour in fire, since FRP materials are known to be combustible and susceptible to deterioration at elevated temperature. Before FRP reinforcement can be used with confidence in buildings, the performance of these materials during fire, and their ability to meet the fire endurance criteria set out in building codes, must be evaluated. To date, information in this area is extremely scarce, and a great deal of further work is required to fill all the gaps in knowledge.

Studies are in progress at the National Research Council of Canada (NRC), in collaboration with Intelligent Sensing for Innovative Structures (ISIS) Canada, Queen's University, and Fyfe Co. LLC, to develop fire resistance design guidelines for FRP-strengthened concrete systems for possible incorporation into codes and standards [1, 3, 4]. As part of this effort, full-scale fire resistance tests were carried out on three FRP-wrapped (confined) reinforced concrete columns. Two of these columns were circular in cross-section while the third was square. The results of these tests are presented in this report. The primary objective of the tests was to investigate the behaviour of FRP-wrapped and insulated reinforced concrete columns under exposure to a standard fire.

TEST SPECIMENS

The experimental program consisted of fire endurance tests on three full-scale reinforced concrete columns strengthened with Tyfo® FRP systems[1]. Two of these columns were circular in cross-section and were designated as Column 1 and Column 2. These two columns were strengthened with the Tyfo® SCH carbon/epoxy FRP system. The third column was designated as Column 7, was square in cross-section, and was strengthened with the Tyfo® SEH glass/epoxy FRP system. Details of the test program are given in Tables 1 and 2.

Dimensions

[1] Certain commercial products are identified in this paper in order to adequately specify the experimental procedure. In no case does such identification imply recommendations or endorsement by the National Research Council, nor does it imply that the product or material identified is the best available for the purpose.

The circular columns were 400 mm in diameter, while the square column was 406 mm in width. All three columns were 3810 mm long from end plate to end plate. The cross-sectional dimensions and reinforcement details of the columns are given in Figure 1.

Materials

Cement

Type I Portland cement, a general purpose cement for construction of reinforced concrete structures, was used for constructing the reinforced concrete columns.

Aggregates

The columns were fabricated with carbonate aggregate concrete. When concrete is made with coarse aggregate consisting mainly of calcium carbonate or a combination of calcium and magnesium carbonate (for example limestone and dolomite), it is referred to as carbonate aggregate concrete. The fine aggregate used was natural sand.

Reinforcement

Deformed bars meeting the requirements of ASTM A615-80 [5] were used for main longitudinal bars, spirals, and ties. All reinforcement had a specified yield strength of 400 MPa. The longitudinal reinforcement in the circular columns was comprised of eight 19.5 mm diameter bars, symmetrically placed, with 40 mm clear cover to the spiral reinforcement. The square column had four 25 mm diameter longitudinal reinforcing bars with 40 mm cover to the ties. The main reinforcing bars were welded to steel end plates. The percentage of longitudinal steel in the circular and square columns was 1.91% and 1.21% respectively. The details of reinforcing bars are given in Figure 1.

The lateral reinforcement for the circular columns consisted of 10 mm diameter deformed steel spiral with a centre-to-centre pitch of 50 mm. The square columns were fabricated with 10 mm diameter deformed square ties spaced at 406 mm. The location and layout of the transverse steel reinforcement are also shown in Figure 1.

Concrete Mix

A single batch of concrete was used for fabricating the two circular columns. The concrete was supplied from Lafarge Ready-Mix, Kingston, Canada, and was delivered to Queen's University. The concrete was non air-entrained, carbonate aggregate concrete with a 28-day compressive strength of 39 MPa. The concrete for the square column was prepared at the NRC Laboratory, Ottawa, and consisted of non air-entrained, carbonate aggregate concrete with a 28-day compressive strength of 52 MPa.

Fabrication

Circular columns were fabricated and cured in the Structures Testing Laboratory at Queen's University in Kingston, and then shipped to NRC for full-scale fire testing. The columns were cast vertically using Sonotube^{TM} formwork. Chromel-alumel thermocouples were secured to the reinforcing steel at specific locations before the cage was precisely positioned in the form. The square column was fabricated and cured in the NRC Fire Laboratories in Ottawa. This column was cast horizontally in plywood formwork. No internal thermocouples were installed in the square column. In order to avoid any possible dislocation of the thermocouples in the circular columns during casting, a careful working plan was followed (for all three columns) as described below.

Reinforcing Bars and Steel Plates

All columns were 3810 mm in length, measured from end plate to end plate. The longitudinal reinforcing bars, were cut to 3790 mm and machined at both ends to the diameters shown in Figure 1.

The details of the steel end plates, including dimensions, are shown in Figure 1. Holes, with a diameter 1.6 mm greater than that of the machined ends, were drilled through the plates to accommodate the longitudinal bars. The main reinforcing bars and lateral ties were tied together to complete the steel cage. The cage was then placed against one end plate in such a way that the machined segments of the bars were positioned in the holes.

Welding

All longitudinal reinforcing bars were welded to the top and bottom end plates. The provisions of AWS Designation D12.1-75 [6] were followed when welding the plates and bars. Special attention was given to the centering and squaring of the end plates during welding. Mild steel welding rods were used to fill the holes, which had been drilled to accommodate the reinforcing bars, on the outer face of the plates. The rough surface of the welded joints on the drilled outer face of the plate was ground to a smooth finish.

The welding of the bottom steel plate was performed before casting the columns. The top plate was placed after pouring and curing of the concrete. Before positioning the top plate, a layer of mortar was spread over the top surface of the column to ensure good contact between the steel plate and the concrete. Using a similar procedure as for the bottom plate, the top plate was welded to the longitudinal reinforcing bars and the outside face was smoothed using a grinder.

Instrumentation

Both circular columns were instrumented with thermocouples and electrical resistance strain gauges at the column mid-height. Figure 2 shows the location and number of various sensors in these columns. Tables 3 and 4 gives a summary of all instrumentation used during the column tests. Square columns had significantly less instrumentation installed during the fabrication process (refer to Table 4). Thermocouples were placed only at the EI-R/VG interface, the VG/FRP interface, and the FRP/concrete interface.

Thermal instrumentation consisted of chromel-alumel (Type-K) thermocouples. Thermocouple frames were fabricated for the thermocouples in the concrete (thermocouples 4 to 7) by fastening individual thermocouples to 3.1 mm diameter steel drill-rod and securely fastening the drill rod to the steel reinforcement cage with wire ties prior to pouring the concrete. Thermocouples 10 to 15 were attached directly to the reinforcing cage using wire ties. Thermocouple wires for sensors placed inside the columns were threaded up the sides of the vertical reinforcing bars inside the reinforcing cage and exited the side of the columns just below the top end-plate.

For thermocouples 2 and 8, at the FRP/concrete interface, shallow vertical grooves were ground into the side of the columns after pouring the concrete from the top-plate to the column mid-height using a hand grinder. The wires for thermocouples 2 and 8 were placed inside the grooves and run down the sides of the columns to the appropriate locations. The grooves were filled with an epoxy patching compound. Thermocouples 1, 9, 16 and 17 were attached to the exterior of the FRP wrap using 5-minute epoxy. Thermocouples 18 and 19 were attached to the columns' surface with wire ties and epoxy before commencing spray application of the VG insulation (described below).

The number and location of strain gauges in the circular columns is depicted in Figure 2. Strain gauges were installed inside the column at its mid-height in an attempt to gain insight into the effectiveness (or lack thereof) of the FRP wrap and to detect any bending in the column during the tests. High temperature strain coupons were fabricated and installed on the longitudinal reinforcing steel at 4 locations, distributed evenly around the column cross-section at mid-height. The strain coupons were fabricated by installing Kyowa™ KFU-5-120-C1-11 high-temperature foil gauges on 150 mm long coupons of 3 mm by 13 mm cold-flat-finished mild steel plate. The gauge lead-wires were soldered to high-temperature polyamide terminal pads using high-temperature silver solder, and high-temperature Kyowa™ Type L-4 strain gauge wire was soldered to the terminal pads. The strain gauge wires were sheathed with protective PVC tubing, which served to provide mechanical protection for the wires while the columns were cast, as well as waterproofing protection inside the hardened concrete. A surface protection and waterproofing layer was applied to the gauge/lead-wire/terminal pad assembly using Superflex™ Red High Temp RTV Silicone Adhesive Sealant. Once fabricated, the metal coupons were tack-welded to the vertical reinforcing bars, and the lead wires ran up the sides of the vertical reinforcing bars and exited the forms just below the top end-plate.

<u>Forms</u>

The formwork for the circular columns consisted of 400 mm inside diameter Sonotube™ formwork. The formwork was tied to the column base plate using metal brackets and tap screws to ensure accurate location of the formwork within the column. An aluminum collar was used at the top of the column to keep the formwork in place. The square column was cast inside custom fabricated plywood forms. Because this column was cast horizontally, the end plates were used to hold the bars in place during casting.

<u>Concrete Placement</u>

The circular columns were poured vertically in a single lift, with concrete supplied by a local ready-mix plant. The concrete was placed using a large hopper attached to an overhead crane, and a series of rubber chutes of various lengths (fabricated specifically for this project to ensure that the concrete never free-fell more than 0.6 m while being placed, and so that the concrete placing operations could be conducted without damaging the instrumentation at the column mid height). The concrete was hand-vibrated during each lift with a 3.65 m long, 32 mm diameter wand vibrator to ensure adequate consolidation.

The square column was poured horizontally in a single lift. This was accomplished by sequentially placing concrete into the horizontal formwork and placing upper formwork panels as the concrete placing process continued. This ensured a perfectly square column. The concrete was vibrated using a wand vibrator during placement to ensure adequate consolidation.

Curing

Once cast, the columns were cured in a humidified plastic enclosure at 21°C to 24°C and 100% relative humidity for seven days, after which point the formwork was removed. The columns were allowed to cure in the laboratory, at ambient temperature and relative humidity, for at least one year until they were ready for wrapping and testing.

FRP Strengthening

The FRP strengthening system for the circular columns consisted of a single layer of the Fyfe Co. Tyfo® SCH unidirectional carbon/epoxy FRP system (SCH) with a Tyfo® S epoxy adhesive/matrix. The wrap design called for a 300 mm overlap in the circumferential direction and a 25 mm overlap in the vertical direction, and resulted in a theoretical ultimate load capacity increase of about 53% based on the ACI 440 [7] design guidelines, and of about 26% based on the ISIS Canada [8] guidelines. Details of load calculations for the columns are presented in Appendices C and D.

The FRP strengthening system for the square column consisted of three layers of the Fyfe Co. Tyfo® SEH unidirectional glass/epoxy FRP system (SEH) with a Tyfo® S epoxy adhesive/matrix. The wrap design called for a 300 mm overlap in the hoop direction and a 25 mm overlap in the vertical direction, and resulted in a theoretical ultimate load capacity increase of about 5% based on the Teng et al. [9] model for rectangular FRP-confined concrete columns, used in conjunction with the ACI 440 [7] design equations for the strength of FRP-wrapped columns. Details of load calculations for the columns are presented in Appendices C and D.

The wrap was installed on the columns in accordance with Fyfe Co. LLC installation procedures. The overall procedure was as follows:

1. The surface of the columns was checked for any major defects or protrusions that could affect the ability of the FRP to bond to the substrate concrete. None were found.
2. The corners of the square column were ground with a concrete grinder to a minimum radius of 25 mm.
3. The surface of the columns was lightly brushed with a heavy-duty scrub brush to remove any dust or loose debris.
4. A thin primer coat, consisting of S-Epoxy mixed with silica-fume, was applied to the entire surface of the column by hand using a trowel. The objective was to fill any surface voids or minor imperfections and to ensure a strong bond between the SCH system and the concrete.
5. The SCH and SEH sheets were cut to the appropriate length and placed on a clean plastic drop sheet where they were saturated with epoxy using a standard paint roller.
6. The SCH and SEH sheets were wrapped around the columns in 610 mm wide lifts, starting at the top and working toward the base of the column. The 300 mm overlapping vertical seams were staggered 90 degrees around the column for each subsequent lift of FRP sheet.
7. Pressure was applied to the FRP sheets by hand to ensure that all air voids were removed from the FRP-concrete interface.
8. The FRP was allowed to cure for at least 16 hours at ambient indoor temperature (approximately 18°C).

Fire Protection

Two similar but unique fire protection systems, developed specifically for this application by Fyfe Co., were used to provide supplemental fire insulation for the columns. The fire protection schemes, called the Tyfo® VG/EI and Tyfo® VG/EI-R systems, consisted of Tyfo® VG insulation (VG) in combination with Tyfo® EI coating (EI) or Tyfo® EI-R coating (EI-R). Details of the insulation system are provided in Figure 3, which shows the details for the circular columns. The insulation details on the square column were similar, with the exception that different thicknesses were used (as specified in Table 4) and EI-R was used instead of EI.

Tyfo® VG insulation is a spray applied cementitious plaster which has an extremely low thermal conductivity and is essentially thermally inert up to temperatures in excess of 1000°C. When exposed to flames, Tyfo® VG plaster releases chemically combined water in the form of water vapor, which helps to maintain the plaster's temperature below 100°C until all of the water has been driven off as steam. Meanwhile, the insulating action of the various fillers delays the release of steam and retards the transmission of heat, thus improving overall fire-proofing characteristics. Tyfo® EI paint is an intumescent epoxy coating, applied by trowel to the exterior of the Tyfo® VG insulation, which expands on heating to form a thick multi-cellular char with a very low-thermal conductivity. Tyfo EI-R is a non-intumescent epoxy paint. Column 1 was protected with 32 mm of VG and 0.56 mm of EI, whereas Column 2 was protected with 57 mm of VG and 0.25 mm of EI. Column 7 was protected with 38 mm of VG and 0.13 mm of EI-R (refer to Figure 3).

VG Insulation Application

To ensure a strong bond between the SCH system and the VG insulation on the circular columns, a single layer of galvanized steel diamond lath was secured to the column over its full height using steel-concrete anchors and brackets. The VG was then spray-applied to the surface of the column in 20 mm lifts until the desired thickness was achieved. Steel lath was not used on the square column. The procedure to apply the VG insulation was as follows:

1. The surface of the FRP wrap was lightly sanded with a coarse-grit sand paper to ensure as strong a bond as possible with the VG.
2. Steel shelving channel sections were attached vertically to the surface of the columns by placing threaded concrete anchors through the FRP wrap and into the substrate concrete. The steel channels were then bolted to the anchors at two locations each.
3. Galvanized steel diamond lath (with 6 mm openings) was attached to the steel shelving channels using conventional metal screws. The diamond lath was applied so as to completely cover the surface of the column such that it would provide reinforcement for the sprayed VG Insulation. Care was taken to ensure that the distance between the surface of the column and the diamond lath was constant.
4. On each column, four circular depth guides were installed at various heights around the column. The depth guides consisted of PVC tubing of known outside diameter, wrapped circumferentially around the diamond lath. The guides were essential to level and screen the surface of the VG after spraying and to obtain a uniform surface.
5. A thin layer of Fyfe Co. Tyfo® VG Primer (VGP) was applied to the surface of the columns using a general-purpose spray bottle.
6. The VG insulation was spray applied in lifts approximately 20 mm thick using an EZ-TEX™ DX Electric Texture Sprayer until the desired overall thickness of VG was achieved.
7. Once the desired thickness of VG had been applied, its surface was hand-toweled to as smooth a finish as possible using drywall trowels. The VG was allowed to cure at room temperature overnight.
8. Once the VG had hardened sufficiently to remain in place while being manipulated, the circumferential depth guides were removed and the resulting grooves were hand-filled with VG using a trowel. The columns were left to dry for four days before application of the first coat of EI.

EI Coating Application

The EI was applied to the exterior surface of the VG by hand using a trowel. The design specifications for the EI material called for 0.51 mm and 0.25 mm thicknesses on columns 1 and 2, respectively. Column 3 was provided with 0.13 mm of EI-R. To achieve these approximate coverages of material, the required volumes of EI were determined based on the surface areas of each of the columns, and the amounts of material actually applied to the columns were monitored throughout the application process.

Based on the volumes of EI applied to the columns, enough material was applied to column 1 for an average coating thickness of 0.56 mm, slightly more than the 0.51 mm specified. For columns 2 and 3 the approximate average thicknesses of EI and EI-R were 0.25 mm and 0.13 mm respectively, as specified in the initial insulation design.

Test Apparatus

The fire endurance experiments were carried out by exposing the columns to heat in a furnace specially built for fire testing loaded columns. The test furnace was designed to produce conditions to which a member might be exposed during a fire, i.e., temperatures, structural loads and heat transfer, and to meet the requirements of ASTM E119 [10]. It consists of a steel framework supported by four steel columns, with the furnace chamber inside the framework (Figure 4). The characteristics and instrumentation of the furnace are described in detail by Lie [11]. Only a brief description of the furnace and the main components are given here.

Loading Device

A hydraulic jack with a capacity of 9778 kN produces a load along the axis of the test column. The jack is located at the bottom of the furnace chamber. The plate on the top of this jack can be used as a platform to which the column can be mounted.

Furnace Chamber

The furnace chamber has a floor area of 2642 x 2642 mm and is 3048 mm high. The interior of the chamber is lined with ceramic insulating materials that efficiently transfer heat to the specimen. The ceiling and floor insulation protect the column end plates from fire. It should be noted that only 3200 mm of the column is exposed to fire.

Heat is supplied by 32 propane gas burners in the furnace chamber, arranged in eight columns containing four burners each. The total capacity of the burners is 4700 kW. Each burner can be adjusted individually, which allows for a high degree of temperature uniformity in the furnace chamber. The pressure in the furnace chamber is also adjustable and was set somewhat lower than atmospheric pressure.

Furnace Instrumentation

The furnace temperatures were measured with the aid of eight Type K chromel-alumel thermocouples. The thermocouple junctions were located 305 mm away from the surface of the test specimen, at various heights. Two thermocouples were placed opposite each other at intervals of 610 mm along the height of the furnace chamber. The locations of their junctions and their numbering are shown in Figure 5. Thermocouples 4 and 6 were located at a height of 610 mm from the floor, Thermocouples 2 and 8 at 1220 mm, Thermocouples 3 and 5 at 1830 mm and Thermocouples 1 and 7 at 2440 mm. The temperatures measured by the thermocouples were averaged automatically and the average temperature was used to control the furnace temperature.

The load was controlled by servocontrollers and measured with pressure transducers. The accuracy of controlling and measuring loads is about 4 kN at lower load levels and relatively better at higher loads.

The axial deformation of the test columns was determined by measuring the displacement of the jack that supported each column. The rotation of the end plates of the columns were determined by measuring the displacement of the plates at a distance of 500 mm from the centre of the hinge, at the top and bottom respectively. The displacements were measured using transducers with an accuracy of 0.002 mm.

TEST CONDITIONS AND PROCEDURES

The columns were installed in the furnace by bolting their end plates to the test frame loading head at the top and the hydraulic jack at the bottom. This resulted in a fixed-fixed end condition for both members, which most accurately simulates the end conditions to be expected in an actual building.

Before testing, the moisture condition at the centre of the columns was measured by inserting a Vaisala moisture sensor into a hole drilled in the concrete. The relative humidity of each column is given in Table 5. These values were used to estimate the moisture content of the concrete using the procedures outlined in ASTM E119 [10].

End Conditions

All columns were tested with both ends fixed, i.e., restrained against rotation and horizontal translation. For this purpose, eight 19 mm (3/4 in) diameter bolts, spaced regularly around the column, were used at each end to bolt the end plate to the loading head at the top and to the hydraulic jack at the bottom.

Loading

All columns were tested under a concentric axial compressive load. The applied load on the circular columns was 2515 kN, which represents 73% of the full design load (factored compressive resistance of the column) determined according to ISIS Design Manual No. 4 [8], or 50% according to ACI 440.2R-02 [7]. The applied load on the square column was 3093 kN, which represents 61% of the full design load (factored compressive resistance of the column) determined according to Teng et al. [9]. The factored compressive resistance of each column, along with the applied loads, are given in Table 5. Full details of the load calculations for the columns are given in Appendices C and D.

In all three tests, loads were applied to the columns approximately 45 min before the start of the fire endurance tests. Loads were maintained until a condition was reached at which no further increase of the axial and rotational deformations could be measured. This condition was selected as the initial condition of the column deformation during the fire test. The load was maintained at a constant value throughout the fire endurance tests.

After 5 hours of fire exposure, the circular columns had demonstrated no signs of impending failure, and so the loads were increased until failure was observed. Actual failure of the circular columns occurred in a sudden and explosive fashion for both columns at about 5.5 hours of fire exposure. The square column failed under its sustained load at just over four hours. Failure loads are also given in Table 5.

Fire Exposure

The ambient temperature at the start of each test was approximately 20°C. During tests, each column was exposed to heating controlled in such a way that the average temperature in the furnace followed, as closely as possible, the ULC S101 [12] standard time-temperature curve, which is equivalent to the ASTM E119 [10] standard fire curve. This curve can be approximately expressed using the following equation:

$$T_f = 20 + 750\left(1 - e^{-3.79533\sqrt{t}}\right) + 170.41\sqrt{t}$$

where: t = time in hours

T_f = temperature of furnace in °C

Recording of Results

The furnace, concrete, steel, insulation and FRP temperatures (where applicable), as well as load, axial deformations of the columns, and strains in the longitudinal reinforcing steel, were recorded at one-minute intervals throughout the fire tests. The fire behaviour of the VG/EI and VG/EI-R insulation and FRP wrap, crack propagation and the occurrence of spalling in columns were also monitored during the test through small observation windows in the column furnace walls. After the completion of fire tests, post-test observations were made to attempt to analyse the failure pattern, extent and nature of spalling and condition of rebars and ties.

Failure Criterion

The columns were considered to have failed, and the tests stopped, if the hydraulic jack, which has a maximum speed of 76 mm/minute, could no longer maintain the sustained load. If this failure criterion was not reached during the first 5 hours of the test, the load was increased until the column failed and the test was stopped after 5.5 hours, beyond which time damage to the test furnace could occur.

RESULTS AND DISCUSSION

The results of the column tests are summarized in Table 5, in which the column characteristics, test conditions, fire endurances, and failure modes are given for each column. The furnace, concrete, steel, insulation, and FRP temperatures recorded during the tests, as well as the axial deformations of the column specimens, are presented graphically in Figures A.1, A.2 and A.3 in Appendix A, where positive axial deformation values indicate expansion of the column. Some typical photos of the FRP-wrapped and insulated columns before and after fire testing are shown in Figures B.1, B.2 and B.3 in Appendix B.

General Observations

Both circular columns were exposed to fire for more than 5 hours without failing under their 2515 kN load. The square column failed under its sustained service load at slightly more than four hours of fire exposure. The most significant visual observations from the tests relate to the performance of the EI and EI-R coatings within the first 20 to 30 minutes of the tests, and to cracking of the VG insulation later in the fire exposure.

<u>Performance of EI Coating</u>

For both circular columns the EI intumescent epoxy coating activated within the first 3 to 4 minutes of the test. Expansion of the EI coating was complete within 10 minutes of fire exposure, and the expanded EI char began to debond from columns' surface in both tests within 15 minutes of exposure. However, the beneficial effects of the EI coating on the overall fire performance of the column are twofold. First, the EI acts as an initial line of defense against fire, and significantly reduces the temperatures in the column in the very early stages of the fire, during the critical shock temperature loading. Second, the EI paint, which is applied to the outer surface of the VG shortly after the VG is applied to the column, acts as a membrane and maintains moisture inside the VG. Thus, the VG has high moisture content when exposed to fire, and its insulating characteristics are greatly enhanced.

<u>Performance of EI-R Coating</u>

For the square column the EI-R non-intumescent epoxy coating burned off within the first 5 to 8 minutes of the test. However, as is the case for the EI coating, the EI-R coating acts as a membrane and maintains moisture inside the VG, causing the VG to have a high moisture content when exposed to fire and enhancing its insulating characteristics.

<u>Performance of VG Insulation</u>

The VG insulation performed extremely well under fire exposure, and remained intact until the end of the tests when explosive concrete spalling caused it to debond. The only change observed in the appearance of the VG insulation during fire exposure was the formation of cracks, generally less than 5 mm wide, which gradually appeared and widened as the test progressed. The location of the cracks appeared to be associated with the thickness of the VG and the location of installation joints in the material. Further tests will be required to determine installation procedures and VG thicknesses that will minimize and/or prevent cracking. The overall timeline and observations recorded during each test are given below.

Column 1

Time hr:min	**Observations**
0:00	Column subjected to predetermined axial load for about 45 min. prior to fire exposure.
0:03	EI epoxy reaction begins; irregular expansion of coating accompanied by flaming.
0:05	Flames observed on surface of EI; epoxy reaction continues.
0:12	EI falls off suddenly and completely; VG exposed to fire.
0:28	Few small cracks (< 5 mm (0.2 in) width) observed in VG at the ring-spacer locations.
0:28-1:30	No significant new observations; slight and gradual (1 to 2 mm (0.04-0.08 in)) widening of cracks in VG.
1:30	Large vertical cracks (≈ 5 mm (0.2 in) width) observed in VG at location of steel channels.
3:03	Flame observed emanating from vertical cracks in VG; assumed to be burning of FRP matrix resin
3:51	Flames emanating from all cracks in VG
3:57-5:00	Hissing sound of moisture evaporation from top of column
5:00	Applied load increased at a rate of approximately 72 kN (16 kips)/min.
5:30	Explosive sounds from inside furnace; spalling of concrete cover; some VG blown off of column; failure of column; fire test halted.
5:34	Furnace doors opened; SCH burning; VG debonded from column in some areas; diamond lath exposed in some areas; some spalling of concrete cover.

Post failure:

Due to the sudden and explosive failure mode observed for the column, much of the VG insulation debonded from the column coincident with failure. This exposed some of the SCH directly to the fire and resulted in burning of the FRP material. Major spalling occurred and some of the longitudinal and spiral steel was exposed. However, no buckling of rebars or deformation of ties occurred in the column.

Column 2

Time (hr:min)	**Observations**
0:00	Column subjected to predetermined axial load for about 45 min. prior to fire exposure.
0:03	Intumescent reaction begins; irregular expansion of coating accompanied by flaming.
0:05	Flames observed on surface of EI; intumescent reaction continues.
0:11	EI falling off in various places; EI surface extremely irregular.
0:13	EI almost completely fallen off; VG exposed to fire.
0:17	Few small cracks (< 5 mm (0.2 in) width) observed in VG.
0:17-4:00	No significant new observations; slight and gradual (1 to 2 mm (0.04-0.08 in)) widening of cracks in VG.
4:00	Average temperature of the SCH system is 101˚C, and it is likely that some deterioration of strength of the SCH FRP composite has occurred.

5:00	Applied load increased at a rate of approximately 87 kN (19.6 kips)/min.
5:30	Explosive sounds from inside furnace; spalling of concrete cover; VG blown off of column; failure of column; fire test halted.
5:34	Furnace doors opened; SCH burning; VG debonded from column over entire height; diamond lath exposed; widespread spalling of concrete cover.

Post failure:

Due to the sudden and explosive failure mode observed for the column, much of the VG insulation debonded from the column coincident with failure. This exposed some of the SCH directly to the fire and resulted in burning of the FRP material for a short period of time (refer to Figure B.2). Major spalling occurred upon failure and some of the longitudinal and spiral steel was exposed. However, no buckling of rebars or deformation of ties was observed.

Column 7

Time (hr:min)	**Observations**
0:00	Column subjected to predetermined axial load for about 45 min. prior to fire exposure.
0:05-0:08	EI-R coating burns off, accompanied by flaming.
0:05	Few small cracks (< 5 mm (0.2 in) width) observed in VG.
0:05- 4:00	No significant new observations; slight and gradual (1 to 2 mm (0.04-0.08 in)) widening of cracks in VG.
4:00	Average temperature of the SEH system is 410˚C, and it is likely that significant deterioration of strength of the SEH FRP composite has occurred.
4:10	Explosive sounds from inside furnace; spalling of concrete cover; VG blown off of column; failure of column; fire test halted.
4:16	Furnace doors opened; SEH burning; VG debonded from column over entire height; diamond lath exposed; widespread spalling of concrete cover.

Post failure:

Due to the sudden and explosive failure mode observed for the column, much of the VG insulation debonded from the column coincident with failure. This exposed some of the SEH directly to the fire and resulted in burning of the FRP material for a short period of time. Major spalling occurred upon failure and the longitudinal and lateral steel were fully exposed. Buckling of rebars and deformation of ties was observed.

Fire Performance

Figures A.1, A.2, and A.3 of Appendix A provide a comparison of recorded temperatures at various locations in columns 1, 2, and 7 during exposure to fire. The data demonstrate that it is possible to maintain the temperature of the FRP wrap below 100°C (212°F) for up to 4 hours by providing the requisite thicknesses of EI and VG insulation materials. Furthermore, based on the temperatures recorded in the concrete and reinforcing steel during fire testing, since temperatures of less than 200ºC (424°F) are not structurally significant in terms of deterioration of mechanical properties for either concrete or steel, it can be stated with confidence that all three columns maintained their full unconfined axial load carrying capacity for greater than 4 hours of exposure to the ASTM e119 standard fire. In addition, both columns were able to carry their FRP-confined service load for the duration of the tests, which would result in a 4 or 5-hour ASTM E119 [10] fire resistance rating (with design loads calculated in accordance with the ISIS Canada guidelines [8] for the circular columns and the Teng et al. [9] procedures for the square column).

Load Capacity and Fire Endurance

The fire endurance of a column is defined as the time to reach failure under exposure to the standard fire [10]. For columns, failure is defined exclusively in terms of load carrying capacity. Thus, the fire endurance of an FRP-wrapped column is defined as the point in time during fire exposure when the load capacity of the member falls below the load to be expected during service [14].

The room temperature axial load capacity of columns 1 and 2 was assumed to be the same, since the VG and EI were assumed to have negligible strength and were not likely to significantly increase the axial load capacity of the columns. The nominal room temperature strength of both columns was calculated to be 5094 kN based on the ISIS Canada [8] design guidelines without reduction factors (or 7054 kN using the ACI 440 Guidelines [7]). Examination of the temperature data obtained during the fire endurance tests leads to the conclusion that, after 5 hours of fire exposure, the wrap had likely been rendered ineffective due to increased temperatures in excess of its glass transition temperature (T_g) but that the concrete and reinforcing steel should have retained virtually all of their room-temperature strength. Thus, when the load was gradually increased after 5 hours of exposure, the columns should have been expected to fail at, or only slightly below, the nominal load for an equivalent unwrapped column at room temperature. The nominal (unfactored) compressive strength of an equivalent unwrapped column was calculated to be 4149 kN using the CSA A23.3-94 [15] code or 4386 kN using ACI 318-95 [16].

Examination of Figure 6, which shows load deflection data for both circular columns (including the preload and fire test phases), shows that columns 2 and 1 had failure loads of approximately 4680 kN and 4437 kN respectively. Figure 7 gives a graphical representation of the load capacities of the two columns with respect to their design and predicted values. In both cases, the tested strength of the columns was similar to the unfactored room temperature predicted strength, calculated according to either the Canadian or American concrete design guidelines. In both fire endurance tests, the columns actually failed at loads slightly higher than those predicted by either code, even after 5 hours of exposure to the ASTM E119 [10] standard fire.

In terms of the fire endurance of the tested circular columns, it can be stated with confidence that, for the full 5.5 hour duration of the tests, both columns were able to carry their FRP-confined (strengthened) service load. This would result in a 5-hour ASTM E119 [10] fire endurance rating with design loads calculated in accordance with the ISIS Canada design guidelines [8]. It is interesting to note also that the unfactored FRP-confined service load calculated according to the ACI 440.2R-02 [7] design guidelines is 2942 kN for the columns tested herein, which is also considerably less than the observed failure load for either column after 5 hours of exposure to the standard fire.

Column 7 failed under its sustained axial service load at just over four hours of exposure to the ASTM E119 [10] standard fire. It is not clear why the performance of this column was not as good as the circular columns, although square columns typically perform more poorly in fire than circular ones. Nonetheless, the square column achieved a 4-hour fire rating under load.

Circular Columns Comparison

Figure 8 provides a comparison of recorded temperatures at various locations in columns 1 and 2. Shown in the figure are average temperature histories at the EI/VG interface, the VG/FRP interface, the FRP/concrete interface, the outside-rebar location, and the column centreline. It is evident that doubling the insulation thickness has a significant effect on the temperatures observed in the columns. Further work is required to determine the precise thicknesses of insulation required to achieve specific levels of fire endurance.

SUMMARY

Based on the results of three full-scale fire endurance tests on two circular and one square FRP-wrapped (confined) reinforced concrete columns, the following points can be summarized:

1. 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 T_g of the polymer matrix.
2. By providing proper fire insulation, as described in this report, a 5 hour fire endurance rating can be achieved for loaded circular reinforced concrete columns strengthened with FRP wraps.
3. By providing proper fire insulation, as described in this report, a 4 hour fire endurance rating can be achieved for loaded square reinforced concrete columns strengthened with FRP wraps.
4. The insulation systems described herein are effective fire protection systems. Visual observations made during the fire tests indicated that the insulation remained intact for more than 4 hours of exposure to the ASTM E119 [10] standard fire.

REFERENCES

1. NRC 1995. National Building Code of Canada 1995. National Research Council of Canada, Ottawa.
2. Kodur, V.K.R. 1999. Fire Resistance Requirements for FRP Structural Members. Proceedings of the Annual Conference of the Canadian Society for Civil Engineering, Regina, Saskatchewan, pp. 83-95.
3. Bisby, L.A., Green, M.F., and Kodur, V.K.R 2001. Fire Behaviour of FRP-Wrapped Reinforced Concrete Columns. In Forde, M.E. (Ed.), Structural Faults and Repair – 2001, London, England, July 4th-6th, CD-ROM.
4. Williams, B.K., Bisby, L.A., Green, M.F., and Kodur, V.K.R. 2003. An Investigation of the Fire Behaviour of FRP-Strengthened Reinforced Concrete Beams, In Forde, M.E. (Ed.), *Structural Faults and Repair – 2003*, London, England, July 1st-3rd, CD-ROM.
5. ASTM 1980. Standard Specification for Deformed and Plain Bullet-Steel Bars for Concrete Reinforcement, ASTM A615-80, American Society for Testing and Materials, West Conshohocken, PA.
6. AWS 1975. Reinforcing Steel Welding Code, AWS- D12.1-75, American Welding Society.
7. ACI 2002. ACI 440.2R-02: Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures. American Concrete Institute, Farmington Hills, MI.

8. ISIS 2001. Strengthening reinforced concrete structures with externally bonded fiber reinforced polymers. Intelligent Sensing for Innovative Structures Canada, Winnipeg, Manitoba.
9. Teng, J., Chen, J., Smith, S., and Lam, L., FRP Strengthened RC Structures, Wiley, UK, 2002, 245 pp.
10. ASTM 2001. Test Method E119-01: Standard Methods of Fire Test of Building Construction and Materials. American Society for Testing and Materials, West Conshohocken, PA.
11. Lie, T.T. 1980. New Facility to Determine Fire Resistance of Columns, Canadian Journal of Civil Engineering, **7**(3), pp. 551-558.
12. CAN/ULC 1989. Standard Methods of Fire Endurance Tests of Building Construction and Materials. CAN/ULC-S101-M89, Underwriters' Laboratories of Canada, Scarborough, ON
13. Lie, T.T. and Celikkol, B. 1991. Method to calculate the fire resistance of circular reinforced concrete columns, ACI Materials Journal, **88**(1), 84-91.
14. Lie, T.T. 1992. Structural Fire Protection. American Society of Civil Engineers Manuals and Reports on Engineering Practice No. 78. ASCE, New York, NY.
15. CSA, 1994. CAN/CSA A23.3-94: Design of Concrete Structures. Canadian Standards Association, Ottawa, Ontario.
16. ACI 1995. Building Code Requirements for Structural Concrete (ACI 318-95). American Concrete Institute, Farmington Hills, MI.

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	404 x 404 mm	3810	4 – 25mm Ø bars	10 mm ties – 406 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*	No. Layers FRP	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 [15]).

Table 3. Instrumentation for the circular column tests[a]

Name	Location[b]	Type	Concrete Cover (mm)
TC1	VG / FRP Interface	Temp. (°C)	-0.76
TC2	FRP / Concrete Interface	Temp. (°C)	0
TC3	Inside Concrete (D/8)	Temp. (°C)	50
TC4	Inside Concrete (D/4)	Temp. (°C)	100
TC5	Inside Concrete (3D/8)	Temp. (°C)	150
TC6	Concrete Centreline	Temp. (°C)	200
TC7	Inside Concrete (D/4)	Temp. (°C)	100
TC8	FRP / Concrete Interface	Temp. (°C)	0
TC9	VG / FRP Interface	Temp. (°C)	-0.76
TC10	Top of Spiral Reinforcement	Temp. (°C)	45
TC11	Outside Edge of Longitudinal Rebar	Temp. (°C)	50
TC12	Inside Edge of Longitudinal Rebar	Temp. (°C)	70
TC13	Inside Edge of Longitudinal Rebar	Temp. (°C)	70
TC14	Outside Edge of Longitudinal Rebar	Temp. (°C)	50
TC15	Top of Spiral Reinforcement	Temp. (°C)	45
TC16	VG / FRP Interface	Temp. (°C)	-0.76
TC17	VG / FRP Interface	Temp. (°C)	-0.76
TC18	EI / VG Interface	Temp. (°C)	-32.5 or -57.9[c]
TC19	EI / VG Interface	Temp. (°C)	-32.5 or -57.9
Strain1	On Longitudinal Rebar 1	Axial Strain (με)	60
Strain2	On Longitudinal Rebar 2	Axial Strain (με)	60
Strain3	On Longitudinal Rebar 3	Axial Strain (με)	60
Strain4	On Longitudinal Rebar 4	Axial Strain (με)	60
FT1	Furnace Interior (2440 mm height)	Temp. (°C)	-505
FT2	Furnace Interior (1220 mm height)	Temp. (°C)	-505
FT3	Furnace Interior (1830 mm height)	Temp. (°C)	-505
FT4	Furnace Interior (610 mm height)	Temp. (°C)	-505
FT5	Furnace Interior (1830 mm height)	Temp. (°C)	-505
FT6	Furnace Interior (610 mm height)	Temp. (°C)	-505
FT7	Furnace Interior (2440 mm height)	Temp. (°C)	-505
FT8	Furnace Interior (1220 mm height)	Temp. (°C)	-505
Load	Total Applied Load (@ base)	Axial Load (kN)	N/A
Disp.	Overall Column Elongation (@ base)	Axial Stroke (mm)	N/A

[a] Refer to Figure 2
[b] All sensors were located at the column mid-height unless otherwise noted
[c] Cover to TCs 18 and 19 depended on the thickness of VG installed

Table 4. Instrumentation for the square column tests[d]

Name	Location[e]	Type	Concrete Cover (mm)
TC1	VG / FRP Interface	Temp. (°C)	-0.76

TC2	VG / FRP Interface	Temp. (°C)	-0.76
TC3	VG / FRP Interface	Temp. (°C)	-0.76
TC4	VG / FRP Interface	Temp. (°C)	-0.76
TC5	FRP / Concrete Interface	Temp. (°C)	0
TC6	FRP / Concrete Interface	Temp. (°C)	0
TC7	VG / FRP Interface	Temp. (°C)	-0.76
TC8	VG / FRP Interface	Temp. (°C)	-0.76
FT1	Furnace Interior (2440 mm height)	Temp. (°C)	-505
FT2	Furnace Interior (1220 mm height)	Temp. (°C)	-505
FT3	Furnace Interior (1830 mm height)	Temp. (°C)	-505
FT4	Furnace Interior (610 mm height)	Temp. (°C)	-505
FT5	Furnace Interior (1830 mm height)	Temp. (°C)	-505
FT6	Furnace Interior (610 mm height)	Temp. (°C)	-505
FT7	Furnace Interior (2440 mm height)	Temp. (°C)	-505
FT8	Furnace Interior (1220 mm height)	Temp. (°C)	-505
Load	Total Applied Load (@ base)	Axial Load (kN)	N/A
Disp.	Overall Column Elongation (@ base)	Axial Stroke (mm)	N/A

[d] Refer to Figure 2
[e] All sensors were located at the column mid-height unless otherwise noted

Table 5. Summary of results of fire endurance tests on Columns 1 and 2

Column	R.H. (%)	Moisture Content [a] (% vol.)	Design Load Capacity [b] (kN)	Applied Load[c] (kN)	Failure Load (kN)	Fire Endurance (hrs)	Failure Mode
Column 1	92.5	7.5	3430	2515	4437	> 5	Crushing/Spalling
Column 2	92.5	7.5	3430	2515	4680	> 5	Crushing/Spalling
Column 7	83.0	6.1	5090	3093	3093	> 4	Crushing/Spalling

[a] Determined in accordance with ULC S101 [12].
[b] Determined in accordance with ISIS Design Manual No. 4 [8] or Teng et al. [9]. Refer to Appendices C and D.
[c] The applied load represents the full unfactored service load, assuming a live-to-dead load ratio of 1:1.

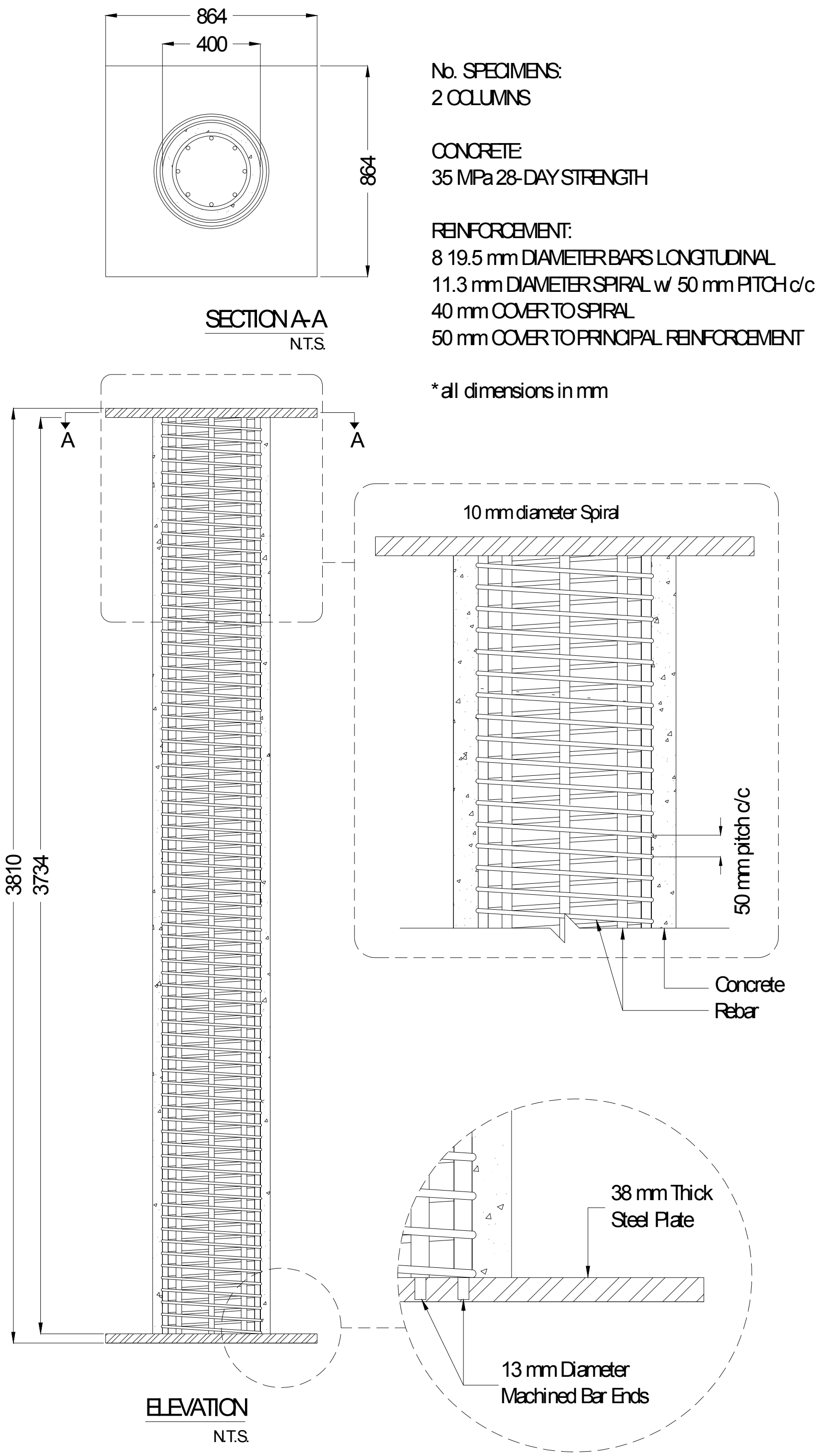


Fig. 1a Elevation and cross-sectional details of Columns 1 and 2

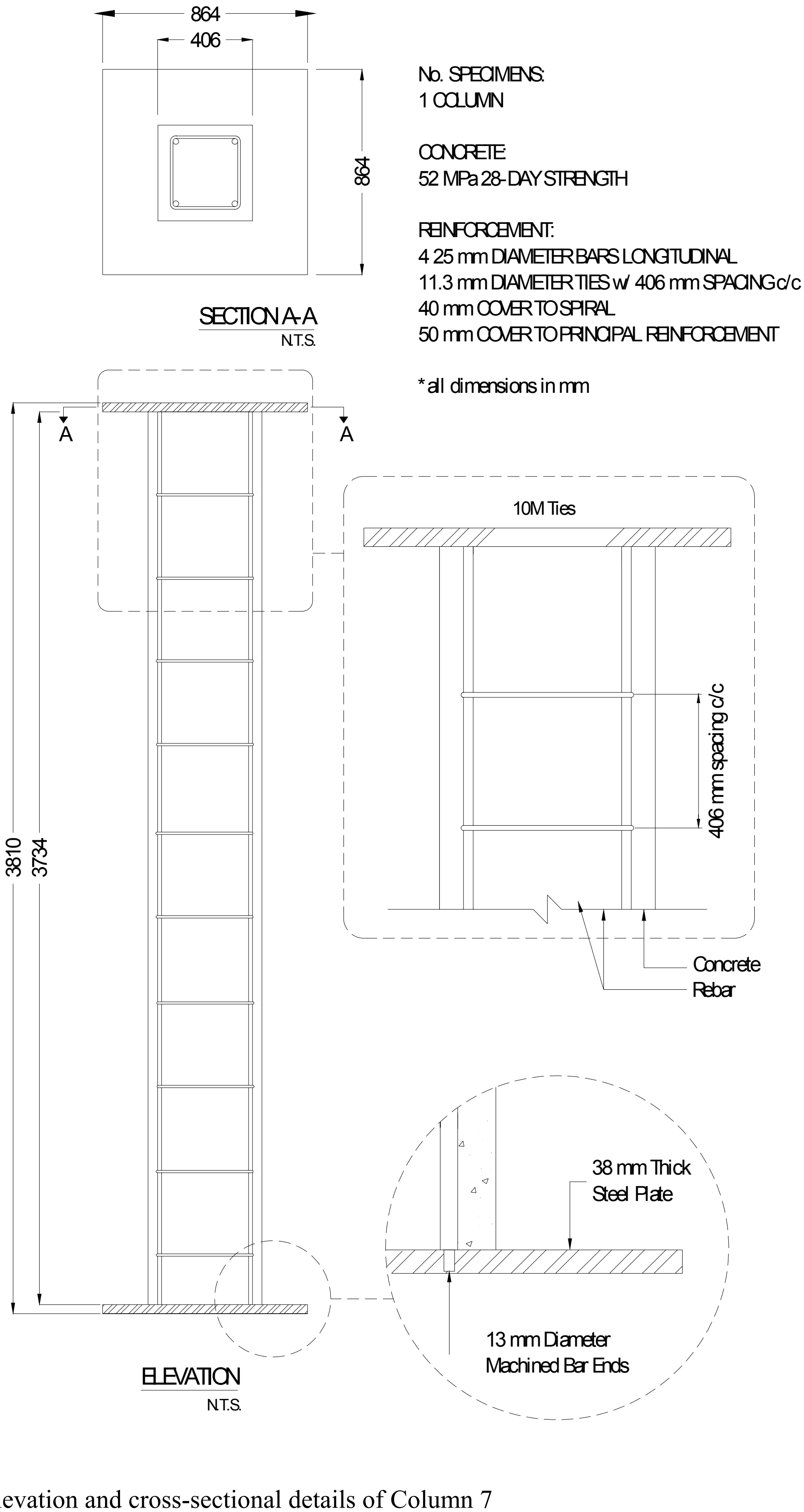


Fig. 1b Elevation and cross-sectional details of Column 7

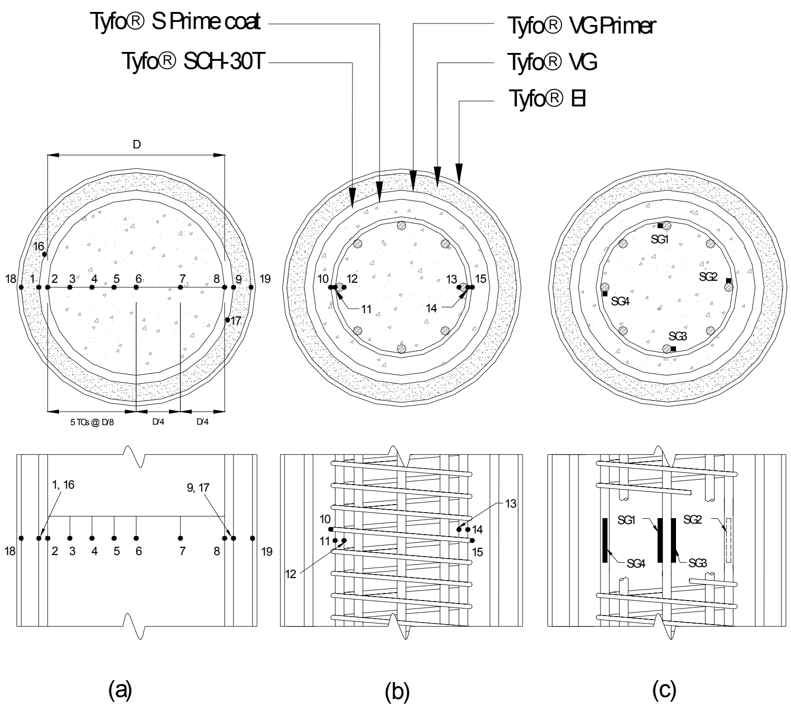


Fig. 2a Location and numbering of thermocouples and strain gauges in columns 1 and 2 at mid-height.
(a) Thermocouples in the concrete and insulation
(b) Thermocouples on the reinforcing steel
(c) Strain gauges on the reinforcing steel

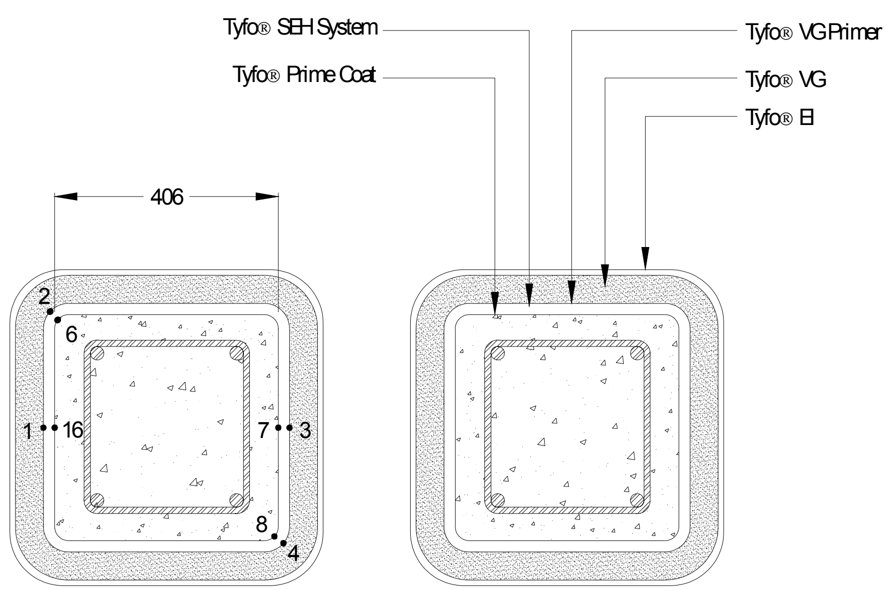
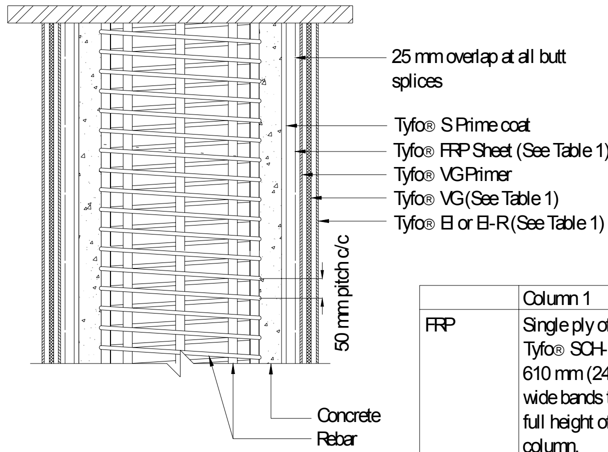


Fig. 2b Location and numbering of thermocouples in column 7 at mid-height.



	Column 1	Column 2	Column 7
FRP	Single ply of Tyfo® SCH-30T 610 mm (24") wide bands to full height of column.	Single ply of Tyfo® SCH-30T 610 mm (24") wide bands to full height of column.	Single ply of Tyfo® SCH-30T 610 mm (24") wide bands to full height of column.
Tyfo® VG	Applied in 2 lifts to the maximum thickness possible; approx 32 mm	Applied in 3 lifts to the maximum thickness possible; approx. 57 mm	Applied in 2 lifts to the maximum thickness possible; approx 32 mm
Tyfo® EI/EI-R	0.56 mm	0.25 mm	0.13 mm

All dimension are in millimiters U.O.S.

Fig. 3 Details of the fire insulation system on columns 1, 2 and 7 (stirrup configuration not representative for column 7)

Fig. 4 NRC column test furnace and Column 2 before testing

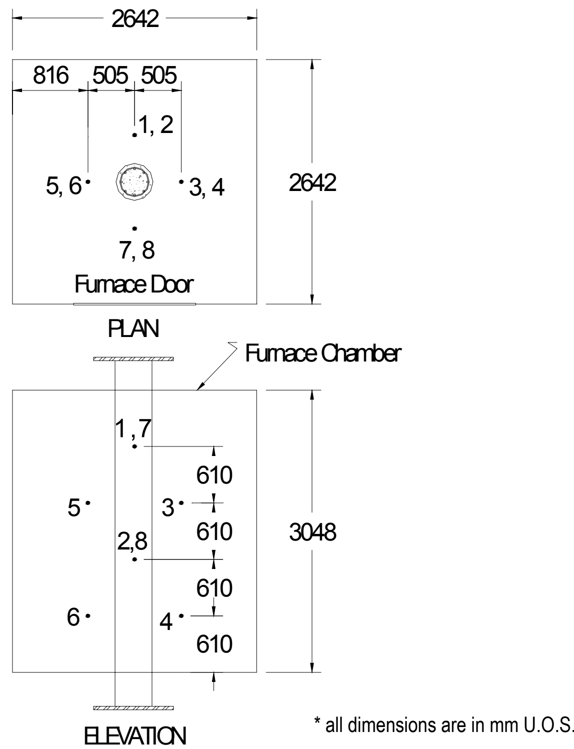


Fig. 5 Location of thermocouples in furnace chamber

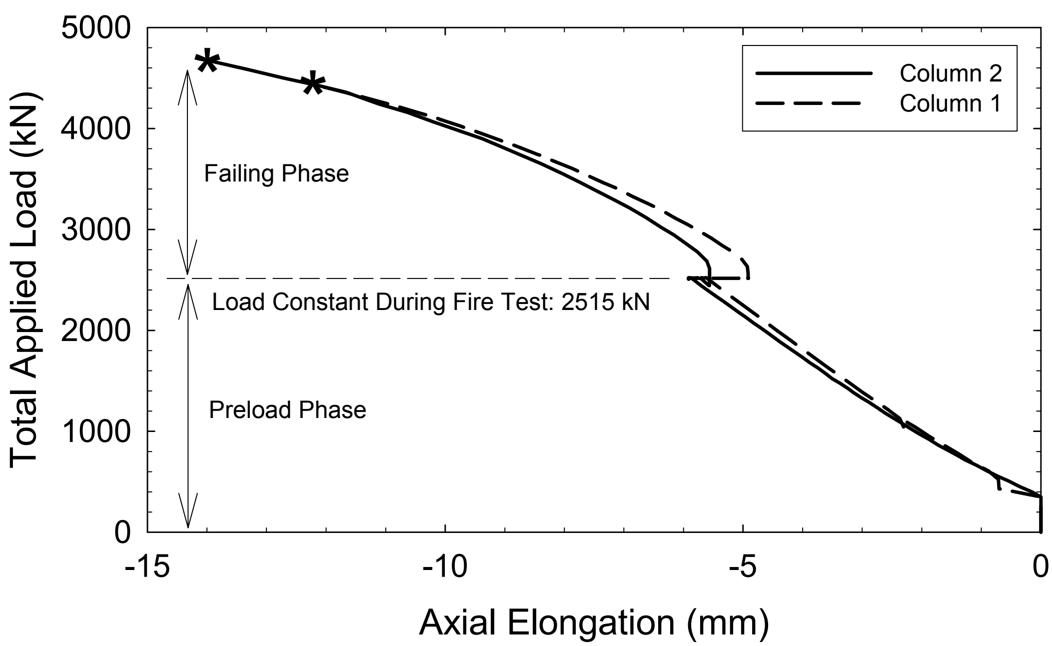


Fig. 6 Load versus deflection data for columns 1 and 2

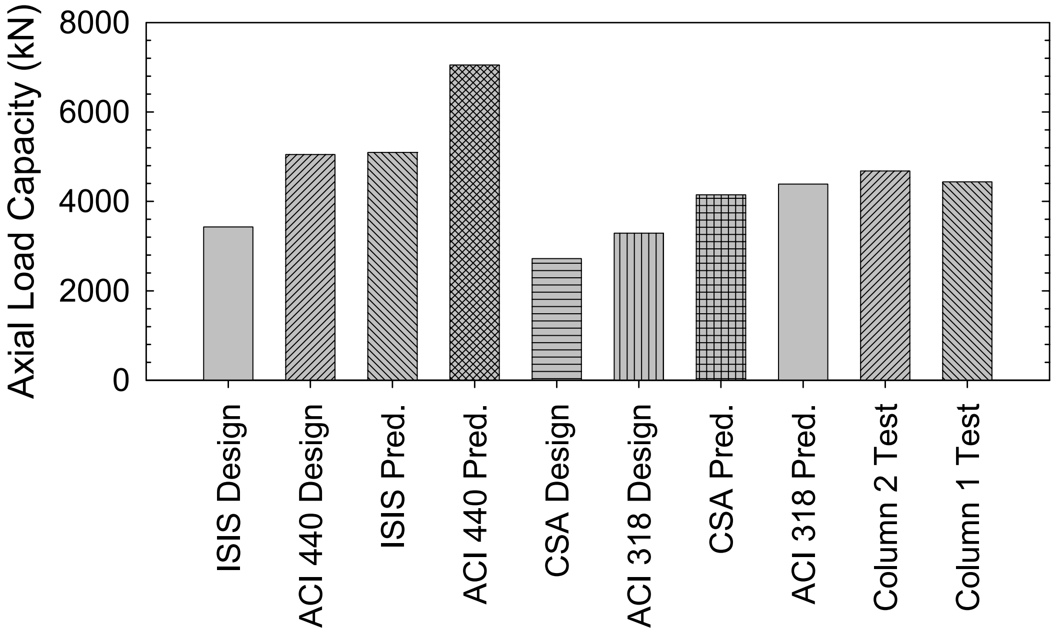
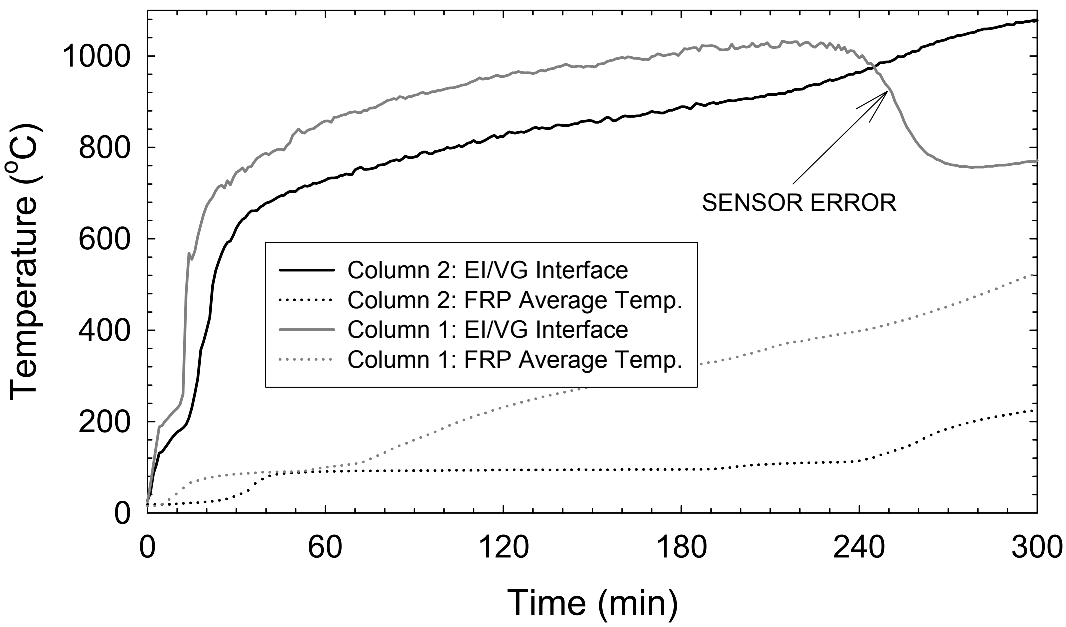
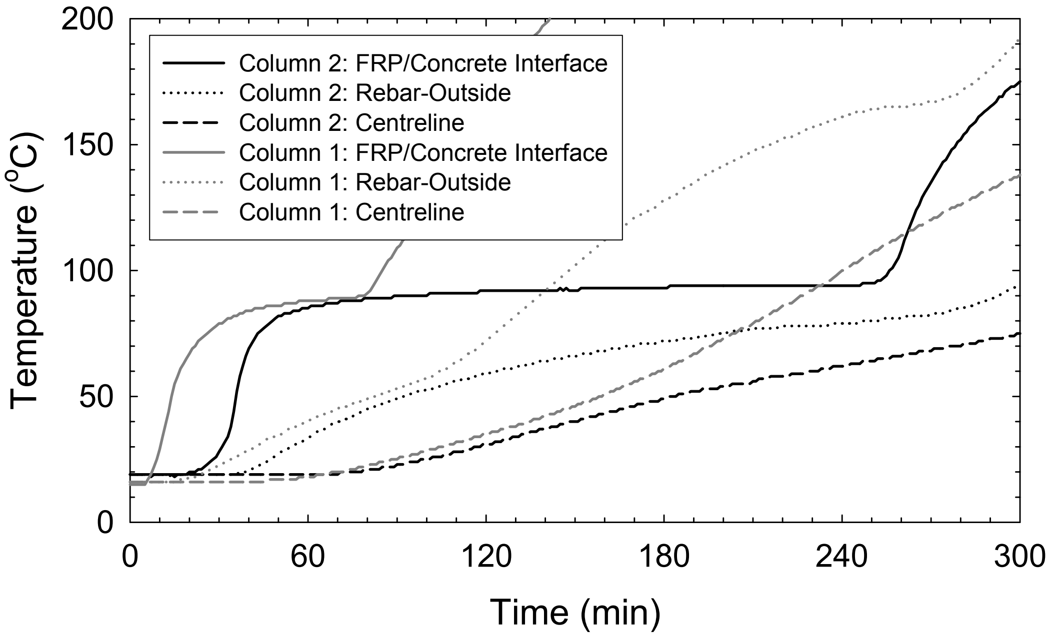


Fig. 7 Load capacities of columns 1 and 2 with respect to their design and predicted values



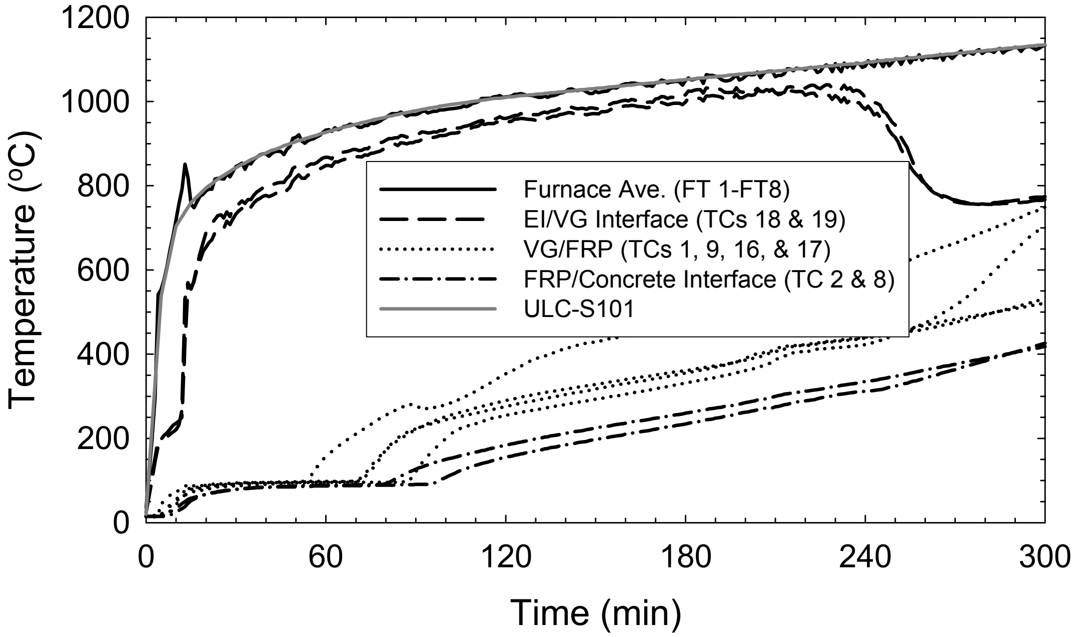
(a)



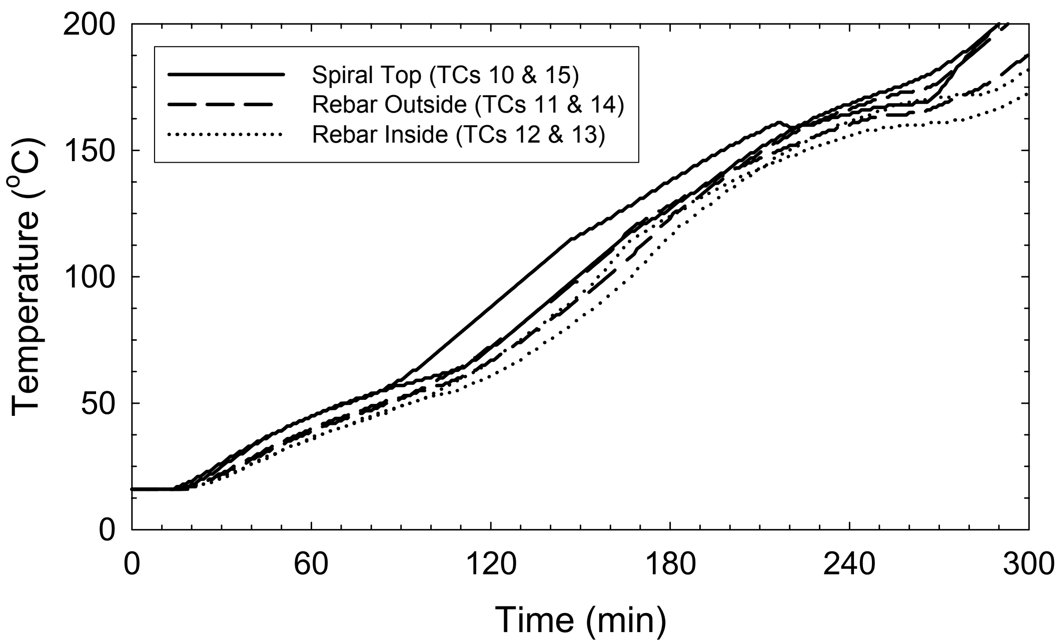
(b)

Fig. 8 Comparison of temperature histories at various locations within columns 1 and 2 during fire endurance tests
(a) Temperatures at the EI/VG and VG/FRP interfaces
(b) Temperatures at the FRP/concrete interface, rebar-outside, and centerline locations

APPENDIX A
TEMPERATURES AND AXIAL DEFORMATIONS OF COLUMNS

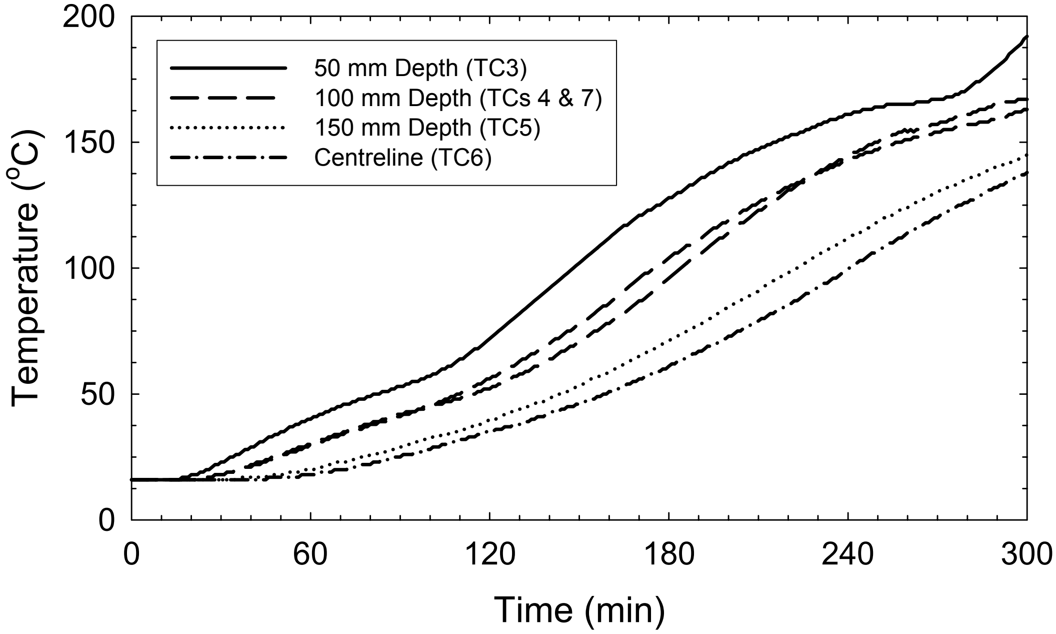


(a)

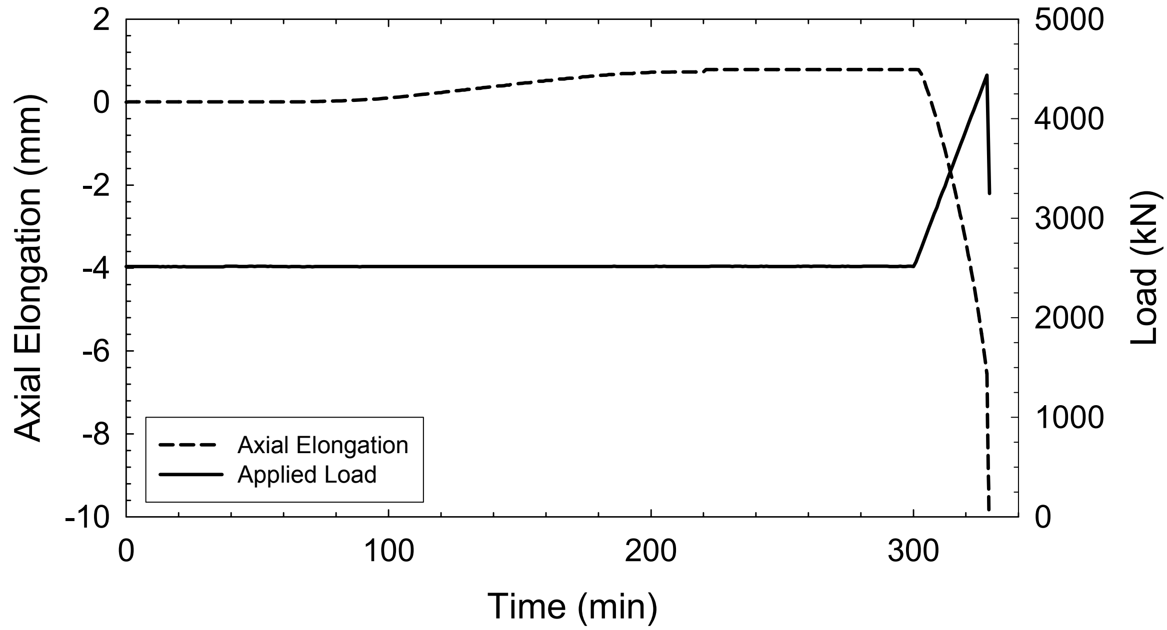


(b)

Fig. A.1 Temperatures and axial deformation for Column 1
(a) Furnace/Insulation/FRP temperatures
(b) Temperatures in reinforcement

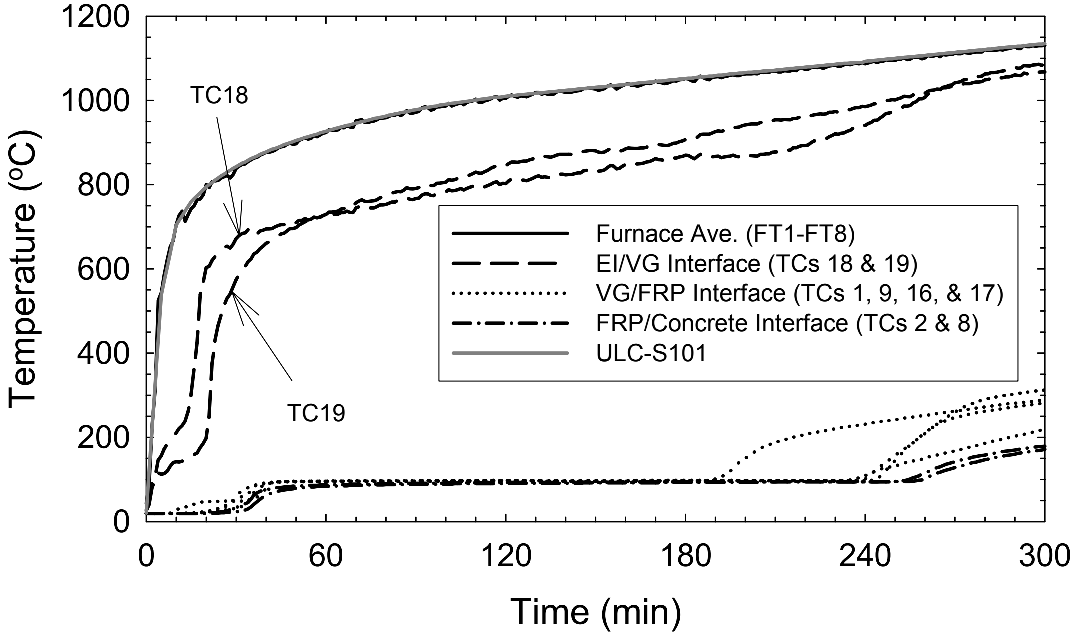


(c)

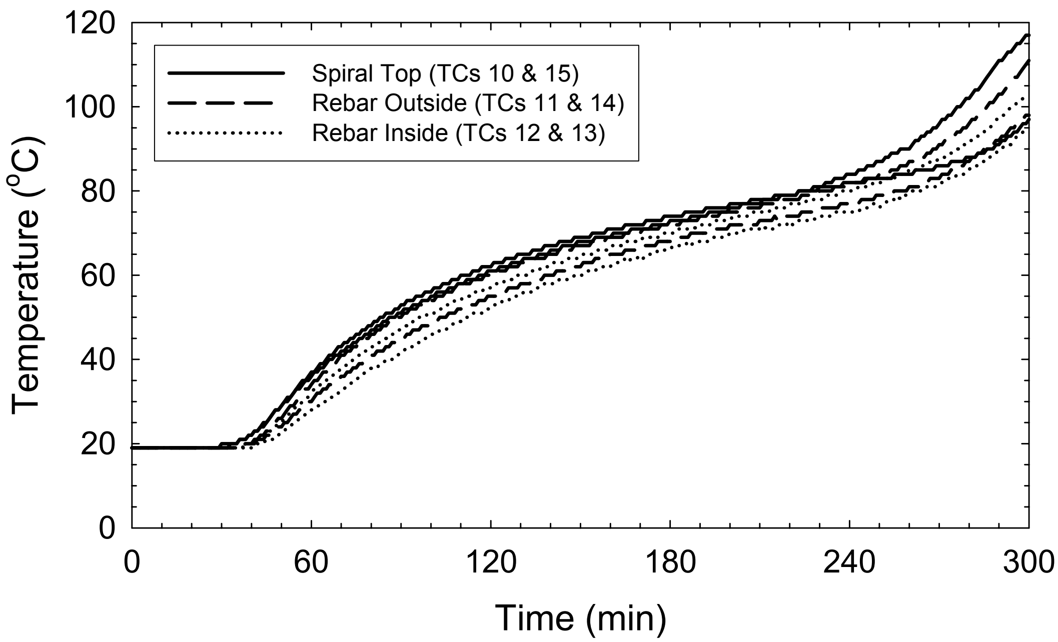


(d)

Fig. A.1 Temperatures and axial deformation for Column 1 (continued)
(c) Temperatures in concrete
(d) Axial deformations

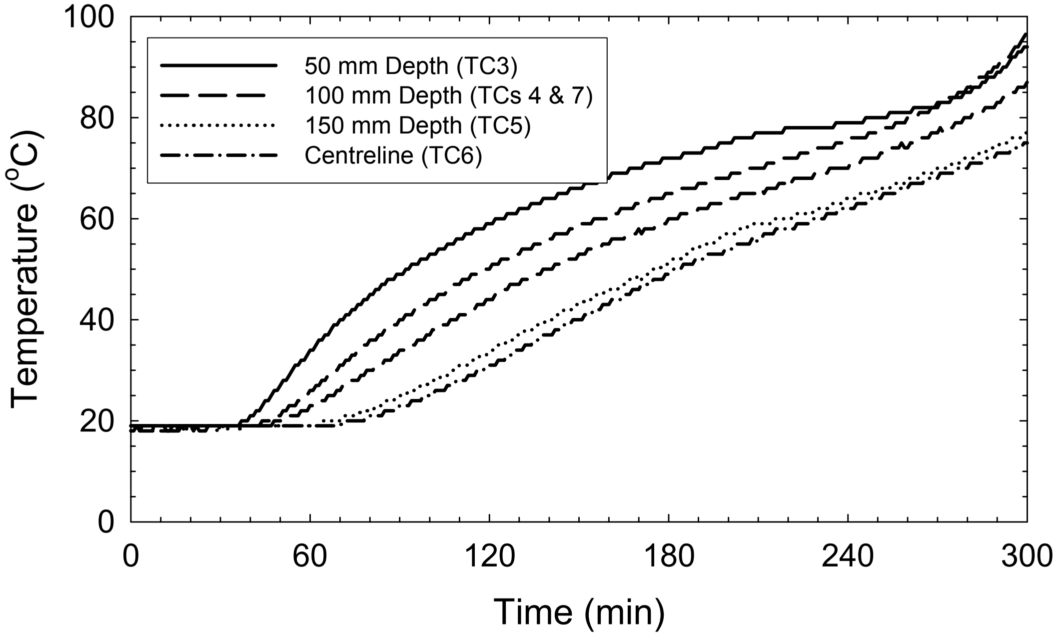


(a)

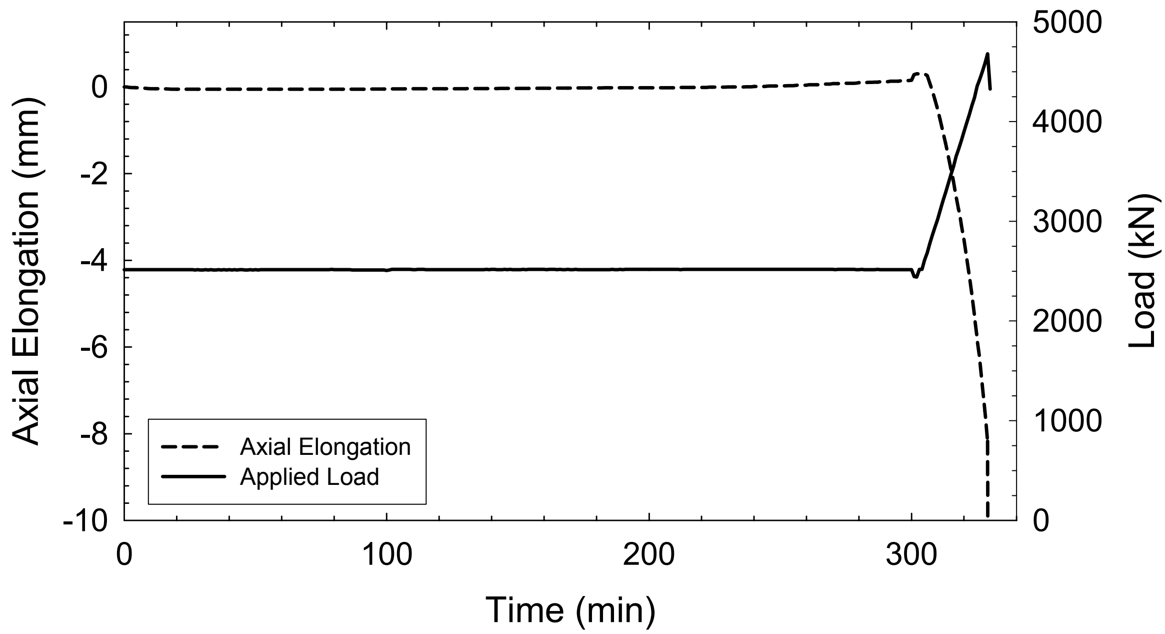


(b)

Fig. A.2 Temperatures and axial deformation for Column 2
(a) Furnace/Insulation/FRP temperatures
(b) Reinforcement temperatures

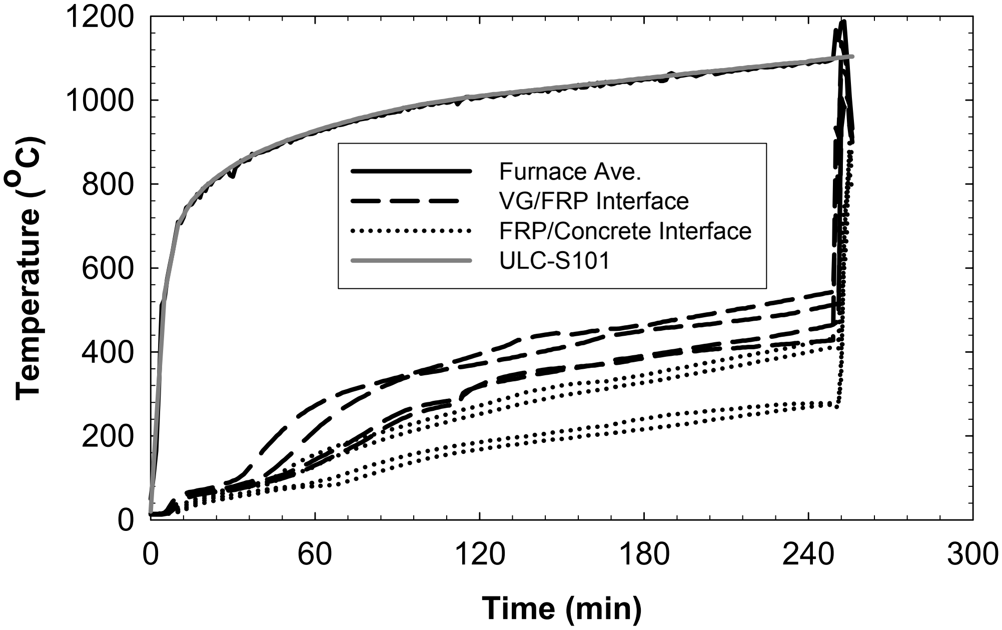


(c)

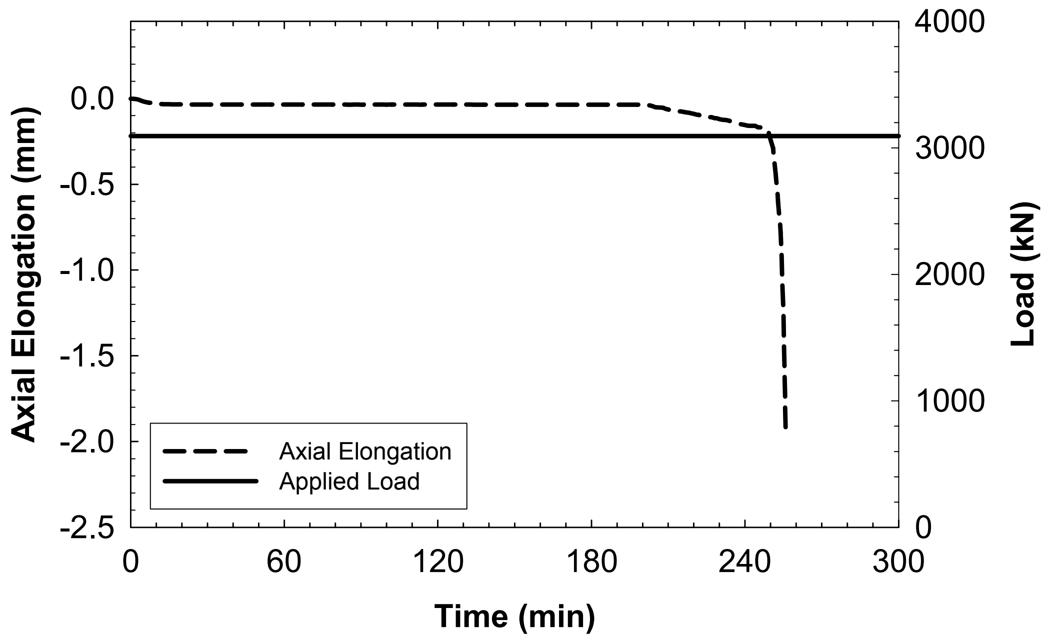


(d)

Fig. A.2 Temperatures and axial deformation for Column 2 (continued)
(c) Concrete temperatures
(d) Axial deformations



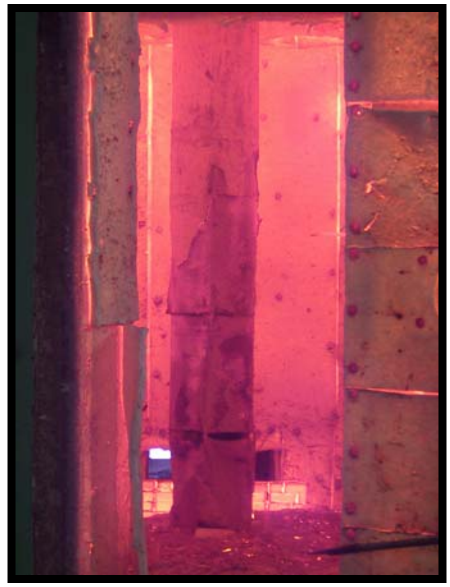
(a)



(b)

Fig. A.3 Temperatures and axial deformation for Column 7
(a) Furnace/Insulation/FRP temperatures
(b) Axial deformations

APPENDIX B
VIEWS OF COLUMN SPECIMENS BEFORE AND AFTER FIRE TESTS

(a) (b)

Fig. B.1 Photos of Column 2
(a) Before fire test
(b) Immediately after fire test


(a) (b)

Fig. B.2 Photos of Column 1
(a) Before fire test
(b) Immediately after fire test

(a) (b)

Fig. B.3 Photos of Column 7
(c) Before fire test
(d) Immediately after fire test

APPENDIX C

LOAD CALCULATIONS FOR CIRCULAR FRP-WRAPPED CONCRETE COLUMNS

This appendix presents detailed load calculations for the circular reinforced concrete columns tested in the study discussed herein. For the unwrapped circular reinforced concrete columns, loads have been calculated using CSA A23.3 [15] and ACI 318 [16]. For calculations relating to FRP-wrapped reinforced concrete columns, calculations have been performed in accordance with the ISIS Canada Design Guidelines [8] and ACI 440.2R-02 [7]. Fire endurance test loads were calculated in accordance with ULC-S101 [12], which is equivalent to ASTM E119 [10].

All calculations have been performed in SI units. Design equations from American codes [7, 10, 16] have been performed using Canadian rebar designations and properties. The actual tested properties have been used where available in design calculations for all materials involved. Tested material properties were as follows:

- Concrete Compressive Strength: $f_c' = 39.5$ MPa
- Yield Strength of Longitudinal Reinforcing Steel: $f_y = 456$ MPa
- Ultimate Tensile Strength of SCH FRP System: $f_{frp,ult} = 1510$ MPa
- Ultimate Strain of SCH FRP System: $\varepsilon_{frp,ult} = 1.64$ %
- Elastic Modulus of SCH FRP System $E_{frp} = 90.2$ GPa

Axial Load Capacity of an Unwrapped Column

ACI 318-95

The maximum axial design strength, $\phi P_{n(\max)}$, for a spirally reinforced concrete column according to ACI 318 [16] is taken as (Cl. 10.3.5.1):

$$\phi P_{n(\max)} = 0.85\phi\left[0.85 f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad [C.1]$$

Where ϕ is equal to 0.75, A_g is the gross cross-sectional area of concrete, and A_{st} is the area of longitudinal reinforcing steel in compression. For the columns tested herein this equation becomes:

$$\begin{aligned}\phi P_{n(\max)} &= 0.85(0.75)\left[0.85(39.5)\left(\pi(200)^2 - 8(300)\right) + 456(8\cdot 300)\right]\\ &= 3289 \text{ kN}\end{aligned}$$

CSA A23.3-94

The maximum design axial load strength in the CSA A23.3 code [15] for a spirally reinforced concrete column is taken as (Cl. 10.10.4):

$$P_{r\max} = 0.85 P_{ro} \qquad [C.2]$$

For the columns considered here we have:

$$P_{ro} = \alpha_1 \phi_c f_c'\left(A_g - A_{st}\right) + \phi_s f_y A_{st} \qquad [C.3]$$

Where ϕ_c is equal to 0.6 and ϕ_s is equal to 0.85. Thus:

$$\begin{aligned} P_{r\max} &= 0.85\left[\alpha_1 \phi_c f_c^{'}\left(A_g - A_{st}\right) + \phi_s f_y A_{st}\right] \\ &= 0.85\left[(0.85 - 0.0015(39.5))(0.6)(39.5)\left(\pi(200)^2 - 8(300)\right) + 0.85(456)(8 \cdot 300)\right] \\ &= 2722 \text{ kN} \end{aligned}$$

Axial Load Capacity of FRP-Wrapped RC Column

ACI 440.2R-02

The FRP-confined design strength for the columns is calculated in accordance with Chapter 11 of ACI 440.2R-02 [7]. The wrap consists of a single layer of SCH FRP, which has the following material properties required for the design calculations:

- Ultimate strength of FRP: f_{com} = 1510 MPa
- FRP Modulus: E_{com} = 90.2 GPa
- Thickness of the wrap: t_w = 0.76 mm

The effective confining pressure in the jacket at ultimate is calculated as follows, with notation modified for consistency within this thesis. The confinement reinforcement ratio, ρ_f, is calculated as:

$$\rho_f = \frac{4 \cdot n \cdot t_w}{d} = \frac{4 \cdot 1 \cdot 0.76}{400} = 0.0076 \qquad \text{[C.4]}$$

The effective ultimate strength of the FRP wrap is taken as the product of the ultimate strength and an environmental reduction coefficient, C_E, as follows:

$$f_{fe} = C_E \cdot f_{com} = 0.85 \cdot 1510 = 1283.5 \text{ MPa} \qquad \text{[C.5]}$$

where C_E is equal to 0.85 for CFRP with an interior conditioned exposure. The confining pressure at ultimate can subsequently be determined as (with κ_a = 1.0 for a circular column):

$$f_l = \frac{\kappa_a \cdot \rho_f \cdot f_{fe}}{2} = \frac{1.0 \cdot 0.0076 \cdot 1283.5}{2} = 4.88 \text{ MPa} \qquad \text{[C.6]}$$

And the confined ultimate strength of the concrete is calculated using the Mander equation:

$$f_{cc}^{'} = f_c^{'} \cdot \left(2.25 \cdot \sqrt{1 + \frac{7.94 \cdot f_l}{f_c^{'}}} - \frac{2 \cdot f_l}{f_c^{'}} - 1.25\right) = 65.1 \text{ MPa} \qquad \text{[C.7]}$$

The ultimate strength of the FRP-wrapped RC column can now be determined as:

$$\begin{aligned} \phi P_{n(\max)} &= 0.85\phi\left[0.85 f_{cc}^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right] \\ &= 0.85 \cdot (0.75) \cdot \left[0.85 \cdot (65.1) \cdot \left(\pi \cdot (200)^2 - 8 \cdot (300)\right) + 456 \cdot (8 \cdot 300)\right] \qquad \text{[C.8]} \\ &= 5049 \text{ kN} \end{aligned}$$

ISIS Canada

According to ISIS Design Manual No. 4 [8], the confining pressure at ultimate is calculated as follows:

$$f_l = \frac{2 \cdot n \cdot \phi_{frp} \cdot f_{com} \cdot t_w}{d} = \frac{2 \cdot 1 \cdot 0.75 \cdot 1510 \cdot 0.76}{400} = 4.30 \text{ MPa} \qquad \text{[C.9]}$$

where ϕ_{frp} is equal to 0.75 for CFRP with an interior conditioned exposure. The ISIS Canada guidelines specify a maximum effective confinement pressure as follows:

$$f_{l(\max)} \leq \frac{f_c^{'}}{2\alpha_{pc}}\left(\frac{1}{k_e} - \phi_c\right) = \frac{39.5}{2 \cdot 1.0}\left(\frac{1}{1.0} - 0.6\right) = 7.8 \text{ MPa} \qquad \text{[C.10]}$$

where α_{pc} = 1.0 and k_e = 1.0 for a round column. The *volumetric confinement ratio* is calculated as:

$$\omega_w = \frac{2 \cdot f_l}{\phi_c \cdot f_c^{'}} = \frac{2 \cdot 4.30}{0.6 \cdot 39.5} = 0.3697 \qquad \text{[C.11]}$$

The confined ultimate strength of the concrete is determined from:

$$f_{cc}^{'} = f_c^{'} \cdot \left(1 + \alpha_{pc}\omega_w\right) = 39.5 \cdot \left(1 + 1.0 \cdot 0.3697\right) = 53.1 \text{ MPa} \qquad \text{[C.12]}$$

Finally, the design strength of the FRP-wrapped column is determined from:

$$\begin{aligned} P_{r\max} &= 0.85\left[\alpha_1 \phi_c f_{cc}^{'}\left(A_g - A_{st}\right) + \phi_s f_y A_{st}\right] \\ &= 0.85 \cdot \left[0.79 \cdot 0.6 \cdot (53.1) \cdot \left(\pi \cdot (200)^2 - 8 \cdot (300)\right) + 0.85 \cdot 456 \cdot (8 \cdot 300)\right] \qquad \text{[C.13]} \\ &= 3430 \text{ kN} \end{aligned}$$

Sustained Load for Fire Endurance Tests

It was felt that the ultimate load capacity for the FRP-wrapped columns as predicted by the ACI design procedure was unconservative. As such, the service load for the fire endurance tests were calculated based on the ISIS Canada design procedures [8]. The ultimate load capacity using the ISIS procedure is calculated above as 3430 kN. If a dead-to-live load ratio of 1:1 is assumed, the service load on the column can be back-calculated using the CSA A23.3 load factors of 1.25 for dead and 1.5 for live. Thus:

$$\begin{aligned} &3430 = 1.5 \cdot S_{LL} + 1.25 \cdot S_{DL} \quad \textit{and} \quad 1.5 \cdot S_{LL} = 1.25 \cdot S_{DL} \\ &\therefore S_{LL} = 1143 \text{ kN} \quad \textit{and} \quad S_{DL} = 1372 \text{ kN} \end{aligned}$$

which results in a service load of 1143 + 1372 = ***2515 kN***. This load was applied to both columns during the fire endurance tests described herein.

Table C.1. Summary of load calculations

Design Document	Unwrapped Ult. Strength (kN)	FRP-Wrapped Ult. Strength (kN)	Strength Increase (kN)	Increase in Strength (%)
ACI 318-95 (F)[a]	3289	--	--	--
CAN/CSA A23.3-94 (F)	2722	--	--	--
ACI 440.2R-02 (F)	3289	5049	1760	53.5
ISIS Canada (F)	2722	3430	708	26.0
ACI 318-95 (U)[b]	4386	--	--	--
CAN/CSA A23.3-94 (U)	4149	--	--	--
ACI 440.2R-02 (U)	4386	7054	2668	60.8
ISIS Canada (U)	4149	5094	945	22.7

[a] F – refers to factored design load calculations (ultimate design capacities, calculated as shown above)
[b] U – refers to unfactored load calculations (predicted load capacities, not shown)

APPENDIX D

LOAD CALCULATIONS FOR SQUARE FRP-WRAPPED CONCRETE COLUMNS

This appendix presents detailed load calculations for the square reinforced concrete column tested in the study discussed herein. For the unwrapped case, loads have been calculated using CSA A23.3 [15] and ACI 318 [16]. For calculations relating to FRP-wrapped reinforced concrete column, calculations have been performed in accordance with the ISIS Canada Design Guidelines [8] and ACI 440.2R-02 [7]. Fire endurance test loads were calculated in accordance with ULC-S101 [12], which is equivalent to ASTM E119 [10].

All calculations have been performed in SI units. Design equations from American codes [7, 10, 16] have been performed using Canadian rebar designations and properties. The actual tested properties have been used where available in design calculations for all materials involved. Tested material properties were as follows:

Column details

- Column cross section = 406.4 mm × 406.4 mm
- Column length = 3810 mm
- Effective length = 1905 mm

Concrete and reinforcement details

- Nominal Area of No. 8 Bars = 509.7 mm^2
- Nominal Area of No. 3 Bars = 71.0 mm^2
- Steel yield stress, $f_y = 413.69$ MPa
- Concrete compressive strength, $f_c' = 52$ MPa

Wrap details (Fyfe Co. LLC TYFO® SEH System)

- Sheet thickness, $t_{frp} = 1.3$ mm
- Ultimate strength, $f_{frp} = 460$ MPa
- Modulus of elasticity, $E_{frp} = 20.9$ GPa
- Strain at Failure, $\varepsilon_{frp} = 2.2\%$

NOTE: Original design by E. Morrow, Fyfe Co. LLC, dated 9/26/03, calls for three layers of SEH-51 system.

Axial Load Capacity of an Unwrapped Column

ACI-318-99

First check if the column is slender (Section 10-8):

$$\frac{kl_u}{r} \leq 34 - \left[12\left(\frac{M_1}{M_2}\right)\right] \qquad \text{[D.1]}$$

From Cl.10.11.1:

$$I = 0.7I_g = 0.7\left(\frac{bh^3}{12}\right) \qquad \text{[D.2]}$$

$$I = 0.7 \times \left(\frac{406.4 \times 406.4^3}{12} \right) = 1.591 \times 10^9 \text{ mm}^4$$

$$A = 1.0 A_g = 1.0 \times (406.4 \times 406.4) = 165.16 \times 10^3 \text{ mm}^2 \qquad [D.3]$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1.591 \times 10^9}{165.16 \times 10^3}} = 98.15 \text{ mm} \qquad [D.4]$$

The column is fixed-fixed, so:

$$kl_u = 0.5 l_u = 0.5 \times 3810 = 1905 \text{ mm} \qquad [D.5]$$

$$\frac{kl_u}{r} = \frac{1905}{98.15} = 19.41 \qquad [D.6]$$

Now:

$$\frac{M_1}{M_2} = 1.0 \text{ for the first test case (equal end moments)}$$

Therefore:

$$\frac{kl_u}{r} = 19.41 < 34 - (12 \times 1.0) = 22$$

and the column is <u>not</u> slender. According to Cl. 10.3.5.2:

$$\Phi P_{n(\max)} = 0.80 \Phi \left[0.85 f_c' \left(A_g - A_{st} \right) + f_y A_{st} \right]$$

$$\Phi P_{n(\max)} = 0.80(0.75)[0.85 \times 52 \times (\{406.4 \times 406.4\} - \{4 \times 509.7\}) + 413.69 \times (4 \times 509.7)] \qquad [D.7]$$

$$\Phi P_{n(\max)} = 4832 \text{ kN}$$

<u>CSA A23.3-94</u>

First check if the column is slender (Cl. 10.15.2):

$$\frac{kl_u}{r} \leq \frac{25 - 10\left(\frac{M_1}{M_2}\right)}{\sqrt{\frac{P_f}{f_c' A_g}}} \qquad [D.8]$$

$$\frac{0.5 \times 3810}{0.3 \times 406.4} \leq \frac{25 - 10(1.0)}{\sqrt{\frac{P_f}{52 \times (406.4 \times 406.4)}}}$$

$P_f \leq 7915$ kN → which is likely true for the column considered here.

Now, from Cl.10.10.4:

$$P_{r\max} = 0.80P_{ro} = 0.80\left[\alpha_1\Phi_c f_c^{'}\left(A_g - A_{st}\right) + \Phi_s A_s f_y\right] \quad \text{[D.9]}$$

where,

$$\alpha_1 = 0.85 - 0.0015 f_c^{'} \geq 0.67 \text{ (Cl.10.1.7(c))} \quad \text{[D.10]}$$

$$\alpha_1 = 0.85 - \left(0.0015 \times 52\right) = 0.772 \geq 0.67$$

$$\Phi_c = 0.6 \text{ (Cl.8.4.2)}$$

$$\Phi_s = 0.85 \text{ (Cl.8.4.3)}$$

Thus:

$$P_{r\max} = 0.80\left[0.772 \times 0.6 \times 52 \times \left(\{406.4 \times 406.4\} - \{4 \times 509.7\}\right) + 0.85 \times \left(4 \times 509.7\right) \times 413.69\right]$$

$$P_{r\max} = 3717 \text{ kN} < 7915 \text{ kN}$$

Therfore, okay.

Axial Load Capacity of FRP-Wrapped RC Column

ACI 440.2R-02 / ICBO Guidelines

There are currently no design guidelines for axial strength increase for square columns, so try using ICBO guidelines:

$$f_{cc}^{'} = f_c^{'}\left(1 + 1.5\rho_j \cos^2\theta\right) \quad \text{[D.11]}$$

where:

$$\rho_j = 2t_j\left(\frac{b+h}{bh}\right) = 2 \times \left(3 \times 1.3\right) \times \left(\frac{406.4 + 406.4}{406.4 \times 406.4}\right) = 0.03839 \quad \text{[D.12]}$$

$$\theta = 0^o \text{ for circumferential wraps.}$$

Therefore:

$$f_{cc}^{'} = 52 \times \left(1 + \left\{1.5 \times 0.03839 \times \cos^2 0^o\right\}\right) = 54.99 \text{ MPa}$$

Now:

$$\Phi P_n = 0.8\Phi\left[0.85\psi_t f_{cc}^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right] \quad \text{[D.13]}$$

$$\Phi P_n = 0.80 \times 0.75 \times \left[0.85 \times 0.95 \times 54.99 \times \left(\{406.4 \times 406.4\} - \{4 \times 509.7\}\right) + 413.69 \times \left(4 \times 509.7\right)\right]$$

$$\Phi P_n = 4852 \text{ kN} \rightarrow 0.41\% \text{ increase only}$$

NOTE: This procedure does not account for corner radius, which is a known performance factor.

ISIS Canada

According to ISIS Design Manual No. 4 [8], we must first check slenderness:

$$\frac{l_u}{h} \leq \frac{7.5}{\sqrt{\dfrac{P_f}{f_c^{'} A_g}}} \text{ (Eqn. 5-12a)} \quad \text{[D.14]}$$

and,

$$\frac{l_u}{h} \leq 10.2 \text{ (Eqn. 5-12b)} \qquad [D.15]$$

$$\frac{1905}{406.4} \leq \frac{7.5}{\sqrt{\frac{P_f}{52 \times 406.4}}} \qquad [D.16]$$

$P_f \leq 54100$ kN → Presumably okay. Check later.

$$f_{frp} = \frac{2N_b \Phi_{frp} E_{frp} \varepsilon_{frp} t_{frp} (b+h)}{bh} \text{ (Eqn. 5-13)} \qquad [D.17]$$

$$f_{frp} = \frac{2 \times 3 \times 0.75 \times 20900 \times 0.002 \times 1.3 \times (406.4 + 406.4)}{406.4 \times 406.4}$$

$$f_{frp} = 1.203 \text{ MPa}$$

$$\omega_w = \frac{f_{lfrp}}{\Phi_c f_c^{'}} = \frac{1.203}{0.6 \times 52} = 0.03856 \text{ (Eqn. 5-14)} \qquad [D.18]$$

$$f_{cc}^{'} = f_c^{'}(1 + \alpha_{pc} \omega_w) \text{ (Eqn. 5-15)} \qquad [D.19]$$

$$f_{cc}^{'} = 52(1 + \{1.0 \times 0.03856\}) = 54 \text{ MPa}$$

Now:

$$P_{r\max} = 0.80[\alpha_1 \Phi_c f_{cc}^{'} (A_g - A_{st}) + \Phi_s A_s f_y] \qquad [D.20]$$

$$P_{r\max} = 0.80[0.772 \times 0.6 \times 54 \times (\{406.4 \times 406.4\} - \{4 \times 509.7\}) + 0.85 \times (4 \times 509.7) \times 413.69]$$

$P_{r\max} = 3838$ kN → 3.26% increase only

<u>Teng et. al. (2002) [9] in conjunction with ACI 440.2R-02</u>

$$\frac{f_{cc}^{'}}{f_c^{'}} = 1 + k_1 k_s \frac{f_l}{f_{co}^{'}} \qquad [D.21]$$

where:

$$k_1 = 2.0$$

and

$$k_s = \frac{b}{h} \frac{A_e}{A_c} \qquad [D.22]$$

$$\frac{A_e}{A_c} = \frac{1 - \frac{\left[\left(b/h\right)(h - 2R_c)^2 + \left(h/b\right)(b - 2R_c)^2\right]}{3A_g} - \rho_{sc}}{1 - \rho_{sc}} \qquad [D.23]$$

where:

$$\rho_{sc} = \frac{A_{st}}{A_g} = \frac{4\times 509.7}{406.4\times 406.4} = 0.01234 \quad \text{[D.24]}$$

$$R_c = 25.4 \text{ mm = corner radius}$$

Therefore:

$$\frac{A_e}{A_c} = \frac{1 - \frac{\left[\left(406.4/406.4\right)\left(406.4-\{2\times 25.4\}\right)^2 + \left(406.4/406.4\right)\left(406.4-\{2\times 25.4\}\right)^2\right]}{3\times\left(406.4\times 406.4\right)} - 0.01234}{1-0.01234}$$

$$\frac{A_e}{A_c} = 0.4832$$

So:

$$k_s = \frac{406.4}{406.4}\times 0.4832 = 0.4832$$

$$f_l = \frac{2 f_{frp} t_{frp}}{\sqrt{h^2+b^2}} = \frac{2\times 460\times 1.3\times 3}{\sqrt{406.4^2+406.4^2}} = 6.24 \quad \text{MPa} \quad \text{[D.25]}$$

$$\frac{f_{cc}^{'}}{f_c^{'}} = f_c^{'}\cdot\left(1+k_1 k_s \frac{f_l}{f_{co}^{'}}\right) = 52\cdot\left(1+2\cdot 0.4832\cdot\frac{6.24}{52}\right) = 58 \quad \text{[D.26]}$$

$$\Phi P_n = 0.8\Phi\left[0.85\psi_f f_{cc}^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right] \quad \text{[D.26]}$$

$$\Phi P_{n\max} = 0.8\times 0.75\times\left[0.85\times 0.95\times 58\times\left(\{406.4\times 406.4\} - \{4\times 509.7\}\right) + 413.69\times\left(4\times 509.7\right)\right]$$

$$\Phi P_n = 5090 \text{ kN} \rightarrow \text{which represents a 5.34\% strength increase}$$

Sustained Load for Fire Endurance Tests

If we take the most severe load condition, or an axial load of 5090 kN, and back-calculate using ACI 318 load factors and assuming a 1:1 dead load to live load ratio, we get:

$$5090 = 1.7S_{L.L} + 1.4S_{D.L}$$
$$1.7S_{L.L} = 1.4S_{D.L}$$

After solving we find:

$$S_{L.L} = 1497 \text{ kN and } S_{D.L} = 1818 \text{ kN}$$

Therefore:

$$S_{service} = 3315 \text{ kN} \rightarrow \text{GOVERNS}$$

Using the ISIS guidelines, we would obtain:

$$5090 = 1.5S_{L.L} + 1.25S_{D.L}$$

$$1.5S_{L.L} = 1.25S_{D.L}$$

Gives:

$$S_{L.L} = 1697 \text{ kN and } S_{D.L} = 2036 \text{ kN}$$

Therefore:

$$S_{service} = 3733 \text{ kN}$$

Thus, use an applied test load of 3733 kN.

Summary

Table D.1. Summary of load calculations

Design Document	Unwrapped Ult. Strength (kN)	FRP-Wrapped Ult. Strength (kN)	Strength Increase (kN)	Increase in Strength (%)
ACI 318-95 (F)[a]	4832	--	--	--
CAN/CSA A23.3-94 (F)	3717	--	--	--
ICBO/ACI 440.2R-02 (F)	4832	4852	20	0.4
ISIS Canada (F)	3717	3838	121	3.3
Teng et al./ ACI 440.2R-02 (F)	4832	5090	258	5.3

[a] F – refers to factored design load calculations (ultimate design capacities, calculated as shown above)