

**FIRE RESPONSE OF REINFORCED CONCRETE BEAMS STRENGTHENED WITH
NEAR-SURFACE MOUNTED FRP REINFORCEMENT**

By

Baolin Yu

A DISSERTATION

Submitted to
Michigan State University
in partial fulfillment of the requirements
for the degree of

Civil Engineering – Doctor of Philosophy

2013

ABSTRACT

FIRE RESPONSE OF REINFORCED CONCRETE BEAMS STRENGTHENED WITH NEAR-SURFACE MOUNTED FRP REINFORCEMENT

By

Baolin Yu

In recent years, the use of near-surface mounted (NSM) fiber-reinforced polymer (FRP) reinforcement has become a promising technology in strengthening of reinforced concrete (RC) structures. When used in buildings, FRP strengthened RC members have to satisfy fire resistance requirements specified in codes and standards. Due to sensitivity of FRP to high temperatures, FRP strengthened RC members usually exhibit relatively low fire resistance. However, NSM FRP strengthening is considered to possess higher fire resistance than traditional externally bonded FRP strengthening. But there are no specific studies on fire response of NSM FRP strengthened RC members. Therefore, experimental and numerical studies were carried out for developing a fundamental understanding on the behavior of NSM FRP strengthened RC beams under fire conditions.

To develop test data on fire response of NSM FRP strengthened members, experimental studies were undertaken at both material level and structural level. As part of material property characterization, extensive high temperature property tests were carried out for evaluating strength, bond, and thermal expansion properties of NSM FRP over a wide temperature range. As part of structural characterization, fire resistance tests were conducted on four NSM FRP strengthened concrete T-beams. Results from these fire tests show that with proper design and configuration, NSM FRP strengthened RC beam can achieve more than three hours of fire resistance, even without fire insulation.

As part of numerical studies, a numerical model was developed for tracing the fire response of NSM FRP strengthened RC beams. The model is based on a macroscopic finite element approach and utilizes moment-curvature relationships to trace the response of beam from pre-loading stage to failure under fire conditions. The model accounts for high temperature properties of constituent materials, various strain components, and fire induced bond degradation. The numerical model was validated using test data generated on various NSM FRP strengthened RC beams at both ambient and fire conditions.

The validated model was further applied to conduct a set of parametric studies to quantify the influence of critical factors on fire response of NSM FRP strengthened RC beams. Results from the studies indicate that type of strengthening, reinforcement ratio of FRP to steel, load level, axial restraint, fire scenario and fire insulation have significant influence on fire resistance of NSM FRP strengthened RC beams. Other factors such as location of NSM FRP and concrete strength have moderate influence on the fire response.

Results from fire experiments and parametric studies were utilized to develop a rational methodology for evaluating the fire resistance of FRP strengthened RC beams. As the first step of this methodology, a set of simplified equations were derived to predict cross sectional temperatures in an FRP strengthened RC beam exposed to fire. Then moment capacity of the strengthened beam is evaluated utilizing an approach similar to that at room temperature but incorporated with temperature dependant strength properties of concrete, steel and FRP. Finally the fire resistance of FRP strengthened RC beam can be determined as the time when external load exceeds moment capacity. This approach facilitates a quick and reliable access on fire resistance of FRP strengthened RC beams, and thus it is attractive for incorporation in design codes and standards.

This dissertation is dedicated to my parents and my wife. Without their emotional support and encouragement, I could not complete this work.

ACKNOWLEDGMENTS

I would like to express my greatest gratitude to my advisor, Professor Venkatesh Kodur, for his continued support, encouragement, and guidance during the course of my studies. I would like to convey my sincere thanks for his ideas and perseverance which made my graduate studies very rewarding.

Also, special thanks to the distinguished faculty members, Prof. Parviz Soroushian, Prof. Lawrence Drzal, and Prof. Nizar Lajnef, who served on my committee and provided me with their valuable advice and useful guidance during my Ph.D. studies.

I would like to thank my friends Anuj Shakya, Esam Aziz, Mohannad Naser, Yi Sun, Nan Hu, Purushutham Pakala, Nikhil Raut, Wasim Khaliq, Aqeel Ahmad, Mahmud Dwaikat, Dr. Xiaomeng Hou and Dr. Haiyan Zhang, for their support, particularly in the experimental part of this study.

I would also like to thank Mr. Siavosh Ravanbakhsh and Mr. Charles Meddaugh for their support and help during the experimental program in this research. Additionally, I would like to thank all the faculty members and students at the Civil and Environmental Engineering department at Michigan State University for their help and support during my doctoral studies.

TABLE OF CONTENTS

LIST OF TABLES.....	xi

LIST OF FIGURES.....	xiv

CHAPTER 1

INTRODUCTION.....	1
1.1 Background and Motivation	1
1.2 Strengthening Strategies for Concrete Structures	4
1.3 Behavior of FRP Strengthened RC Beams under Fire Conditions	7
1.4 Objectives.....	10
1.5 Scope.....	11

CHAPTER 2

STATE-OF-THE-ART REVIEW.....	14
2.1 General.....	14
2.2 Configuration and Installation of NSM FRP Strengthening	15
2.2.1 NSM FRP reinforcement and groove filler.....	15
2.2.2 Installation procedure.....	17
2.3 Behavior of NSM FRP Strengthened Members at Ambient Conditions	21
2.3.1 Bond behavior of NSM FRP system.....	21
2.3.2 Behavior of NSM FRP strengthened RC members	24
2.4 Material Properties at Elevated Temperatures.....	25
2.4.1 Concrete	26
2.4.1.1 Thermal properties.....	26
2.4.1.2 Mechanical properties	29
2.4.1.3 Deformation properties.....	32
2.4.1.4 Fire induced spalling	34
2.4.2 Reinforcing steel	36
2.4.2.1 Thermal properties.....	36
2.4.2.2 Mechanical properties	37
2.4.2.3 Deformation properties.....	39
2.4.3 FRP reinforcement	40
2.4.3.1 General	40
2.4.3.2 Thermal properties.....	42
2.4.3.3 Mechanical properties	44

2.4.3.4 Deformation properties.....	46
2.4.3.5 Bond properties.....	48
2.4.4 Fire insulation	52
2.5 Fire Response of Concrete Beams Incorporated with FRP Reinforcement ...	55
2.5.1 Concrete beams reinforced with interal FRP rebars	55
2.5.2 RC beams strengthened with external FRP laminates	60
2.5.3 RC beams strengthened with NSM FRP reinforcement	66
2.6 Codes and Standards for FRP Strengthened RC members	67
2.7 Summary	69
 CHAPTER 3	
HIGH TEMPERATURE MATERIAL PROPERTY.....	71
3.1 General	71
3.2 Tensile Strength Tests	71
3.2.1 Preparation of test specimens.....	72
3.2.2 Test set-up.....	75
3.2.3 Results and discussion	78
3.2.4 Relations for tensile strength and modulus with temperature.....	87
3.2.5 Summary of tension test results	91
3.3 Bond Strength Tests.....	91
3.3.1 Preparation of test specimens.....	92
3.3.2 Test set-up.....	95
3.3.3 Results and discussion	97
3.3.3.1 Bond strength and modulus at room temperature	97
3.3.3.2 Bond strength and modulus at elevated temperature	101
3.3.3.3 Bond stress-slip relations.....	107
3.3.4 Relations for bond strength and modulus with temperature	110
3.3.5 Summary of bond test results.....	114
3.4 Thermal Expansion Tests	114
3.4.1 Preparation of test specimens.....	115
3.4.2 Test apparatus and test procedure	116
3.4.3 Results and discussion	117
3.4.4 Summary of thermal expansion tests	121
3.5 Summary	122
 CHAPTER 4	
FIRE RESISTANCE EXPERIMENTS.....	124
4.1 General	124

4.2 Preparation of Test Specimens	124
4.2.1 Design and fabrication of RC T-beams	125
4.2.2 NSM FRP strengthening	128
4.2.2.1 Design of flexural strengthening	128
4.2.2.2 Installation of NSM FRP strips	129
4.2.3 Fire insulation on T-beams	131
4.2.3.1 Fire insulation properties	131
4.2.3.2 Installation of fire insulation	132
4.2.4 Instrumentation	134
4.3 Test Apparatus	135
4.4 Test Conditions and Procedure	137
4.5 Material Tests	138
4.6 Test Results and Discussion	140
4.6.1 Test observations	140
4.6.2 Thermal response	144
4.6.2.1 Furnace temperatures	144
4.6.2.2 NSM FRP temperatures	145
4.6.2.3 Steel rebar temperatures	148
4.6.2.4 Concrete temperatures	151
4.6.3 Structural response	153
4.6.3.1 Deflections	153
4.6.3.2 Axial restraint force	156
4.6.3.3 Strain in longitudinal reinforcement	158
4.6.4 Fire resistance	158
4.7 Residual Strength Tests of NSM FRP Strengthened RC Beams	160
4.7.1 Test procedure	161
4.7.2 Results and discussion	161
4.8 Summary	165
 CHAPTER 5	
NUMERICAL MODEL	167
5.1 General	167
5.2 Macroscopic Finite Element Model for Fire Resistance Analysis	167
5.2.1 General approach	168
5.2.2 Fire temperatures	171
5.2.3 Thermal analysis	171
5.2.4 Structural analysis	175
5.2.4.1 General analysis procedure	175

5.2.4.2 Evaluating temperature induced slip and axial restraint force	177
5.2.4.3 Generation of moment-curvature ($M-\kappa$) relationships	183
5.2.4.4 Beam analysis	185
5.3 Computer Implementation	188
5.3.1 Input data	188
5.3.2 Output results	190
5.3.3 Material properties	190
5.4 Validation of Numerical Model	192
5.4.1 Response at ambient conditions	192
5.4.2 Response under fire conditions – Rectangular beams	196
5.4.3 Response under fire conditions – T-beams	200
5.5 Summary	209
 CHAPTER 6	
PARAMETRIC STUDIES.....	211
6.1 General	211
6.2 Critical Factors Influencing Fire Resistance	211
6.3 Parametric Studies	212
6.3.1 Beam configuration and parameters in study	212
6.3.2 Material properties	215
6.3.3 Discretization and analysis details	218
6.3.4 Failure criteria	218
6.4 Results of Parametric Studies	219
6.4.1 Effect of FRP strengthening	221
6.4.2 Effect of NSM FRP location	226
6.4.3 Effect of reinforcement ratio of FRP and steel rebar	228
6.4.4 Effect of concrete compressive strength	231
6.4.5 Effect of load level	233
6.4.6 Effect of axial restraint	234
6.4.7 Effect of fire scenario	238
6.4.8 Effect of insulation layout	241
6.5 Summary	245
 CHAPTER 7	
RATIONAL DESIGN METHODOLOGY.....	247
7.1 General	247
7.2 Simplified Approach for Predicting Temperatures in RC Members	248
7.2.1 An approach for predicting temperature in an uninsulated RC member ...	248

7.2.1.1 General	248
7.2.1.2 Generation of temperature data for regression analysis	250
7.2.1.3 Cross section division for 1-D and 2-D heat transfer area	252
7.2.1.4 Nonlinear regression analysis	256
7.2.1.5 Regression analysis results	259
7.2.1.6 Verification of temperature equations using test results	262
7.2.1.7 Verification of temperature equations using FEA results.....	269
7.2.2 An approach for predicting temperatures in an insulated RC member.....	275
7.2.2.1 Converting fire insulation layer to equivalent concrete layer	275
7.2.2.2 Regression analysis.....	280
7.2.2.3 Verification of temperature equations using test results	284
7.2.2.4 Verification of temperature equations using FEA results	288
7.3 Evaluating Moment Capacity of FRP-Strengthened RC Beams	293
7.3.1 Degradation of steel and FRP properties	293
7.3.2 Effective concrete width under fire exposure	294
7.3.3 Evaluating moment capacity at a given fire exposure time	296
7.4 Validation of the Proposed Approach	300
7.5 Limitation of Applicability	305
7.6 Summary	305
 CHAPTER 8	
CONCLUSIONS AND RECOMMENDATIONS.....	307
8.1 General	307
8.2 Key Findings	308
8.3 Recommendations for Future Research.....	311
8.4 Research Impact.....	312
 APPENDICES.....	315
APPENDIX A Material Properties at Elevated Temperatures	316
APPENDIX B Design and Load Calculations	335
APPENDIX C Finite Element Formulation	341
APPENDIX D Design Examples	343
 REFERENCES.....	355

LIST OF TABLES

Table 2.1 Thermal expansion of FRP reinforcement reported in previous studies	49
Table 2.2 Comparison of thermal properties for different fire insulation.....	55
Table 2.3 Experimental studies on fire response of concrete beams reinforced with internal FRP rebars	59
Table 2.4 Numerical studies on fire response of concrete beams reinforced with internal FRP rebars.....	59
Table 2.5 Experimental studies on fire response of RC beams strengthened with external FRP laminates	64
Table 2.6 Numerical studies on fire response of RC beams strengthened with external FRP laminates	65
Table 2.7 Experimental studies on fire response of RC beams strengthened with NSM FRP reinforcement	65
Table 3.1 Properties of NSM CFRP reinforcement as specified by manufacturer	72
Table 3.2 Tensile strength and elastic modulus of CFRP strips at various temperatures .	81
Table 3.3 Tensile strength and elastic modulus of CFRP rods at various temperatures...	82
Table 3.4 Bond test program on NSM FRP system.....	92
Table 3.5 Bond strength and modulus of Tyfo T300 epoxy for NSM CFRP strip at various temperatures	99
Table 3.6 Bond strength and modulus of Tyfo T300 epoxy for NSM CFRP rod at various temperatures	99
Table 3.7 Bond strength and modulus of Tyfo S epoxy for NSM CFRP strip at various temperatures	100
Table 3.8 Bond strength and modulus of Tyfo S epoxy for NSM CFRP rod at various temperatures	100
Table 3.9 NSM FRP specimens used for thermal expansion test.....	115

Table 3.10 Transverse and longitudinal CTEs for various NSM FRP reinforcement	122
Table 4.1 Batch proportion of concrete	126
Table 4.2 Properties of Tyfo NSM CFRP strips	129
Table 4.3 Variables studied in fire tests on NSM FRP strengthened T-beams.....	138
Table 4.4 Compressive strength of concrete.....	139
Table 4.5 Visual observation for Beams I and II in the first fire resistance test.....	142
Table 4.6 Visual observation for Beams III and IV in the second fire resistance test....	143
Table 4.7 Configuration and test conditions of RC beams with various FRP strengthening	153
Table 4.8 Test variables and results in residual strength tests on fire exposed beams ...	161
Table 5.1 Strain components in concrete, steel, and FRP.....	186
Table 5.2 Configuration and properties of RC beams used for validation	194
Table 6.1 Geometric and material properties of FRP strengthened RC beams used in parametric study.....	213
Table 6.2 Critical factors investigated in parametric study	217
Table 6.3 Summary of fire resistance values for the beams in parametric studies	220
Table 6.4 Configuration and moment contribution of NSM FRP and steel rebar in Beams III 1-3	230
Table 6.5 Effect of insulation layout on fire response of NSM FRP strengthened beams	242
Table 7.1 Characteristics of RC members for regression analysis	252
Table 7.2 Sections of RC members used in validation of temperature equations	264
Table 7.3 Characteristics of insulated RC beams used for the regression analysis	281
Table 7.4 Factors for calculating effective concrete width for various RC beams exposed to ASTM E119 standard fire.....	295

Table 7.5 Comparison of fire resistance using proposed approach against fire tests and FEA results	302
Table A.1 Values for main parameters of the stress-strain relationships of NSC at elevated temperature (Eurocode 2)	321
Table A.2 Values for main parameters of stress-strain relationships of reinforcing steel at elevated temperatures (Eurocode 2)	324
Table A.3 Previous studies on thermal properties of epoxy	329
Table D.1 Properties of Beams D1 and D2.....	344

LIST OF FIGURES

Figure 1.1 Application of NSM FRP on concrete members (For interpretation of the references to color in this and all other figures, the reader is referred to the electronic version of this dissertation)	3
Figure 1.2 Comparison between EBR and NSM strengthening systems under bending....	6
Figure 1.3 Comparison of temperature in NSM FRP and external FRP under standard fire	10
Figure 2.1 Procedures of installing NSM FRP (Hughes Brothers 2011).....	18
Figure 2.2 Requirement on dimensions of NSM groove (ACI 440.2R 2008).....	20
Figure 2.3 Typical failure modes of NSM FRP system.....	22
Figure 2.4 Typical bond-slip curve of NSM FRP system.....	23
Figure 2.5 Variation of thermal properties with temperature for various types of concrete	27
Figure 2.6 Variation of compressive strength with temperature for various types of concrete (Kodur et al. 2008)	30
Figure 2.7 Variation of elastic modulus with temperature for various types of concrete.	31
Figure 2.8 Variation of residual strength of concrete with temperature	32
Figure 2.9 Variation of thermal strain with temperature for various types of concrete....	33
Figure 2.10 Variation of thermal properties with temperature for reinforcing steel	37
Figure 2.11 Variation of yield strength and ultimate strength with temperature for reinforcing steel	38
Figure 2.12 Variation of thermal expansion with temperature for reinforcing steel	40
Figure 2.13 Variation of thermal properties with temperature for FRP	43
Figure 2.14 Variation of bond strength with temperature for externally bonded FRP	51
Figure 2.15 Variation of thermal properties with temperature for VG insulation	54

Figure 3.1 Fabrication of anchor system for FRP specimens	74
Figure 3.2 Test apparatus and specimens for room temperature test.....	75
Figure 3.3 Test setup for FRP tension test at elevated temperatures	76
Figure 3.4 Temperature progression in FRP during high temperature tension tests	77
Figure 3.5 Comparison of measured stresses using loading cell with strain gauges	78
Figure 3.6 Variation of tensile strength and elastic modulus of CFRP strips with temperature	83
Figure 3.7 Variation of tensile strength and elastic modulus of CFRP rods with temperature	83
Figure 3.8 Stress-strain response of CFRP strips at various temperatures	85
Figure 3.9 Stress-strain response of CFRP rods at various temperatures	85
Figure 3.10 Failure modes of CFRP strips at various temperatures	86
Figure 3.11 Failure modes of CFRP rods at various temperatures	87
Figure 3.12 Comparison of tensile strength predicted by empirical formula with test data	90
Figure 3.13 Comparison of elastic modulus predicted by empirical formula with test data	90
Figure 3.14 Fabrication of NSM FRP bond test specimen	92
Figure 3.15 Groove size for installation of NSM FRP specified in ACI 440.2 (2008)	94
Figure 3.16 Test set-up for evaluating bond strength of NSM systems at high temperatures	96
Figure 3.17 Variation of bond strength and elastic modulus of NSM CFRP strip and rod with Tyfo T300 epoxy with temperature	102
Figure 3.18 Variation of bond strength and bond modulus of NSM CFRP strip and rod with Tyfo S epoxy with temperature	103
Figure 3.19 Variation of temperature inside Tyfo T300 and Tyfo S epoxy as a function of heating time.....	104

Figure 3.20 Failure modes of NSM CFRP specimens with Tyfo T300 epoxy.....	106
Figure 3.21 Failure modes of NSM CFRP specimens with Tyfo S epoxy	107
Figure 3.22 Bond stress-slip relations for NSM CFRP specimens with Tyfo T300 epoxy at various temperatures	108
Figure 3.23 Bond stress-slip relations for NSM CFRP specimens with Tyfo S epoxy at various temperatures	109
Figure 3.24 Comparison of predicted bond strength from proposed empirical relations with measured data from tests.....	113
Figure 3.25 Comparison of predicted bond modulus from proposed empirical relations with measured data from tests.....	113
Figure 3.26 TMA apparatus and setup for thermal expansion test.....	117
Figure 3.27 Thermal expansion of NSM FRP specimens in transverse directions	118
Figure 3.28 Thermal expansion of NSM FRP specimens in longitudinal directions	119
Figure 4.1 Elevation, cross-section, and instrumentation of FRP strengthened RC beams	127
Figure 4.2 Steps in fabrication of RC beams	128
Figure 4.3 Location and dimensions of NSM grooves (Units: mm).....	131
Figure 4.4 Installation of NSM FRP strengthening on RC T-beams.....	131
Figure 4.5 Steps in application of fire insulation on NSM FRP strengthened RC beams	133
Figure 4.6 Layout of fire insulation scheme on NSM FRP strengthened RC beams	134
Figure 4.7 Structural fire test furnace at MSU Civil and Infrastructure Laboratory	136
Figure 4.8 Installation of axial restraint on NSM FRP strengthened RC beam (Beam III)	136
Figure 4.9 Stress-strain relations of steel rebars used for flexural reinforcement	139
Figure 4.10 Measured and specified time-temperature curve during fire tests.....	144

Figure 4.11 Variation of NSM FRP temperatures with fire exposure time in Beams I-IV	147
Figure 4.12 Variation of temperatures at insulation/concrete interface with fire exposure time in Beams II-IV	148
Figure 4.13 Variation of steel rebar temperatures with fire exposure time in Beams I-IV	150
Figure 4.14 Variation of concrete temperatures with fire exposure time at various locations in Beams I-IV	152
Figure 4.15 Comparison of mid-span deflections of NSM FRP strengthened RC beams with unstrengthened RC beam and external FRP strengthened RC beam.....	156
Figure 4.16 Variation of axial force and displacement with fire exposure time.....	157
Figure 4.17 Strain measured in tension and compression rebars in Beams I and II during the test (starting from pre-loading stage)	159
Figure 4.18 Strain measured in tension and compression rebars in Beams III and IV during the test (starting from pre-loading stage).....	159
Figure 4.19 Load-deflection response of Beams I-IV in residual strength tests.....	163
Figure 4.20 Failure patterns of Beams I-IV in residual strength tests	164
Figure 4.21 Response of NSM FRP strips after failure in residual strength tests	165
Figure 5.1 Typical beam layout and discretization of beam into segments and elements	169
Figure 5.2 Flowchart illustrating the steps associated in the numerical model	170
Figure 5.3 Bond stress-slip relations of NSM FRP strip at various temperatures	178
Figure 5.4 Force equilibrium at NSM FRP-concrete interface in the <i>ith</i> segment (vertical view)	179
Figure 5.5 Illustration of axial restraint force calculations	182
Figure 5.6 Force equilibrium and strain compatibility in an RC beam strengthened with NSM FRP	184
Figure 5.7 Illustration of curvature controlled iterative procedure for beam analysis....	188

Figure 5.8 Configuration of tested beams for room temperature response validation (Units: mm)	193
Figure 5.9 Load-deflection response in RC beams under monotonic loading (ambient condition)	196
Figure 5.10 Configuration of tested beams for fire condition response validation (Units: mm)	197
Figure 5.11 Comparison of predicted and measured temperatures and mid-span deflections for Beams V4 and V5	200
Figure 5.12 Configuration of tested T-beams for fire condition response validation (Units: mm)	201
Figure 5.13 Comparison of predicted and measured temperatures in NSM FRP and steel rebar for MSU beams	203
Figure 5.14 Comparison of predicted and measured temperatures in concrete for MSU beams	205
Figure 5.15 Comparison of predicted and measured mid-span deflections in T-beams	208
Figure 5.16 Comparison of predicted and measured axial forces in T-beams	209
Figure 6.1 Configuration and elevation of NSM FRP strengthened RC beam (Beam A) for parametric study (Units: mm)	214
Figure 6.2 Layout of NSM FRP strengthened RC beam and discretization along beam length and cross section	219
Figure 6.3 RC beams analyzed for studying the effect of FRP strengthening (Unit: mm)	221
Figure 6.4 Effect of FRP strengthening type on temperature rise in steel rebar and FRP	223
Figure 6.5 Effect of FRP strengthening type on the variation of moment capacity of RC beams	224
Figure 6.6 Effect of FRP strengthening type on the variation of mid-span deflection of RC beams	225
Figure 6.7 RC beams analyzed for studying the effect of NSM FRP location (Units: mm)	226

Figure 6.8 Effect of FRP location on temperatures rise in FRP	227
Figure 6.9 Effect of FRP location on the variation of moment capacity of NSM strengthened RC beams.....	228
Figure 6.10 Effect of reinforcement ratio of FRP and steel rebar on the variation of moment capacity of NSM strengthened RC beams	231
Figure 6.11 Effect of concrete compressive strength on the variation of moment capacity of NSM strengthened RC beams.....	232
Figure 6.12 Effect of load level on the variation of mid-span deflections of NSM strengthened RC beams.....	234
Figure 6.13 Effect of axial restraint on the variation of mid-span deflections of NSM FRP strengthened RC beams.....	236
Figure 6.14 Illustration of axial restraint force under fire conditions.....	236
Figure 6.15 Variation of axial force in NSM FRP strengthened RC beams as a function of fire exposure time	238
Figure 6.16 Standard and design fire temperature curves used in parametric study	239
Figure 6.17 Effect of fire exposure on temperature rise in corner FRP strip.....	240
Figure 6.18 Effect of fire exposure on the variation of mid-span deflections in NSM FRP strengthened RC beams.....	241
Figure 6.19 RC beams analyzed for studying the effect of fire insulation scheme	243
Figure 6.20 Effect of insulation thickness on temperature rise in NSM FRP strips.....	244
Figure 6.21 Effect of insulation depth on temperature rise in NSM FRP strips.....	245
Figure 7.1 Variation of temperature with depth from the bottom of an RC beam at various times (section 300×500mm)	253
Figure 7.2 Variation of temperature with distance from the side surface of an RC beam at various times (section 300×500mm).....	254
Figure 7.3 Cross section idealization for heat transfer analysis in concrete members exposed to different fire conditions	256
Figure 7.4 Comparison of predicted temperatures from the proposed equations with those from FEA	260

Figure 7.5 Validation of the proposed approach by comparing predicted and measured temperatures for NSC-CA members	265
Figure 7.6 Validation of the proposed approach by comparing predicted and measured temperatures for HSC-CA members	267
Figure 7.7 Validation of the proposed approach by comparing predicted and measured temperatures for NSC-SA members	268
Figure 7.8 Validation of the proposed approach by comparing predicted and measured temperatures for HSC-SA members	269
Figure 7.9 Validation of the proposed approach by comparing predicted temperatures with FEA results for NSC-CA members	271
Figure 7.10 Validation of the proposed approach by comparing predicted temperatures with FEA results for HSC-CA members	272
Figure 7.11 Validation of the proposed approach by comparing predicted temperatures with FEA results for NSC-SA members	273
Figure 7.12 Validation of the proposed approach by comparing predicted temperatures with FEA results for HSC-SA members	274
Figure 7.13 Illustration of the equivalent concrete depth method	277
Figure 7.14 FRP strengthened RC beams used in FEA for regression and validation (Units: mm).....	281
Figure 7.15 Comparison of predicted temperatures from the proposed equations (Eqns. 7.18-7.22) with those from FEA (Beam 200×300mm)	283
Figure 7.16 Comparison of predicted temperatures from the proposed equations (Eqns. 7.18-7.22) with those from FEA (Beam 250×400mm)	283
Figure 7.17 Comparison of predicted temperatures from the proposed equations (Eqns. 7.18-7.22) with those from FEA (Beam 300×500mm)	284
Figure 7.18 Validation of the proposed approach by comparing predicted and measured temperatures (Blontrock et al. 2000)	286
Figure 7.19 Validation of the proposed approach by comparing predicted and measured temperatures (Williams et al. 2008).....	286

Figure 7.20 Validation of the proposed approach by comparing predicted and measured temperatures (Palmieri et al. 2012)	287
Figure 7.21 Validation of the proposed approach by comparing predicted and measured temperatures (MSU Beam II).....	287
Figure 7.22 Validation of the proposed approach by comparing predicted temperatures with FEA results (Beam 200×350mm with Aestver insulation).....	289
Figure 7.23 Validation of the proposed approach by comparing predicted temperatures with FEA results (Beam 200×350mm with VG insulation)	290
Figure 7.24 Validation of the proposed approach by comparing predicted temperatures with FEA results (Beam 350×500mm with Aestver insulation).....	291
Figure 7.25 Validation of the proposed approach by comparing predicted temperatures with FEA results (Beam 350×500mm with VG insulation)	292
Figure 7.26 Force equilibrium and strain compatibility of NSM FRP strengthened RC beam at a given fire exposure time	296
Figure 7.27 A flowchart illustrating rational design approach for evaluating fire resistance of FRP strengthened beam	300
Figure 7.28 Validation of the proposed approach by comparing predicted moment capacity with FEA results (Beam 200×350mm with VG insulation)	303
Figure 7.29 Validation of the proposed approach by comparing predicted moment capacity with FEA results (Beam 350×500 mm with VG insulation).....	304
Figure B.1 Cross section, elevation and internal force diagram of RC T-beam.....	337
Figure B.2 Configuration of NSM FRP strengthened RC T-beam.....	339
Figure C.1 Q4 element in transformed coordinates	342
Figure D.1 Layout and cross section of NSM FRP strengthened RC beam (Beam D1)	343
Figure D.2 Variation of temperatures in steel rebar and NSM FRP with fire exposure time in Beam D1	346
Figure D.3 Variation of moment capacity of Beam D1 with fire exposure time.....	348
Figure D.4 Layout and cross section of external FRP strengthened RC beam (Beam D2)	349

Figure D.5 Variation of temperatures in steel rebar and external FRP with fire exposure time in Beam D2	351
Figure D.6 Variation of moment capacity of Beam D2 with fire exposure time.....	354

CHAPTER 1

INTRODUCTION

1.1 Background and Motivation

Concrete is one of the widely used construction materials in civil construction. Concrete structures experience deterioration over a long time, due to poor maintenance, corrosion of steel reinforcement, as well as aging of concrete. Moreover, older concrete structures are often needed to be strengthened to resist extreme loading events such as blast, earthquake, etc. Therefore, in recent years, retrofitting deteriorated or damaged concrete structures has become an increasingly urgent task for civil engineers and stakeholders. Based on a recent “Report Card for America’s infrastructure” released by American Society of Civil Engineers (ASCE 2013), the United States has made no significant progress for more than a decade in improving either the conditions of roads, bridges, power plants, or other vital infrastructure. Estimated investment on repairing the nation’s infrastructure has grown to a daunting \$3.6 trillion over the next ten years. Additional repair and retrofitting costs of seismically deficient structures, deteriorating civil and military infrastructure may run into additional billions of dollars annually.

In order to retrofit concrete structures efficiently and economically, a number of innovative techniques for repairing and rehabilitation of reinforced concrete infrastructures have been developed and implemented, and the most notable one is through the use of fiber reinforced polymer (FRP) laminate as external flexural or shear strengthening. Initially developed for aerospace and automotive industries, FRP has

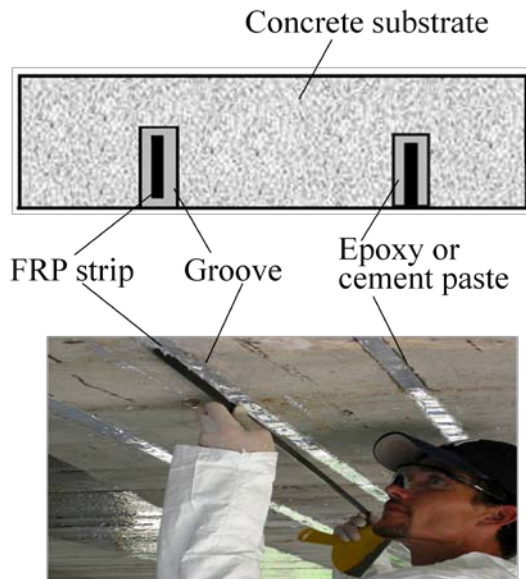
become a promising material for reinforcing and strengthening of concrete infrastructures. This is attributed to numerous advantages of FRP over other traditional materials (steel or concrete), such as high strength to weight ratio, excellent resistance to corrosion, low conductivity, and high fatigue resistance. Therefore, FRP has been increasingly used in civil infrastructures, over a wide range of configurations for external strengthening and reinforcing of masonry walls, for seismic retrofitting of bridges, and as internal reinforcement in power plant and offshore structures.

In the last decade, there have been some advances in FRP strengthening techniques for civil infrastructures. In addition to external FRP strengthening and internal FRP reinforcement, an innovative strengthening technique, near-surface mounted (NSM) FRP strengthening, is gradually gaining popularity. In this technique, an FRP strip or rod is inserted into a pre-cut groove on the concrete cover of an RC member, and then filling the groove with an epoxy adhesive or cementitious grout, as shown in Figure 1.1. The adhesive or grout in the groove ensures that FRP strip or rod is well-anchored inside to concrete and acts as an effective tensile or shear reinforcement in resisting loading on the concrete members. Compared to other strengthening techniques, such as externally bonded reinforcing method (EBR), NSM strengthening can utilize more of the strength of FRP because of better bond adherence (Barros et al. 2007, Oehlers et al. 2008, Rashid et al. 2008). Thus, NSM FRP strengthening is becoming an attractive strengthening method in retrofitting of structures.

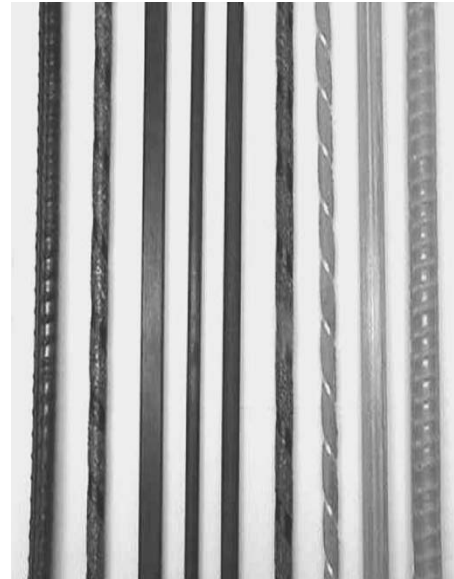
Until now, the application of FRP strengthening is mainly limited to bridges and exterior structures, where fire resistance of the structural members is not a primary concern. It has been established that FRP materials are highly combustible when

subjected to heat flux. The released heat, smoke, and toxic gases during burning of FRP can significantly increase severity of fire. Also, the strength and stiffness of FRP decrease considerably at high temperatures, and the bond between FRP and concrete also degrades quickly due to melting of epoxy resin. Thus fire response is always a concern for FRP strengthened RC members.

When used in buildings, the provision of appropriate fire resistance to structural members is a major design requirement. So far there are limited studies on the fire response of NSM FRP strengthened RC structures, and a large number of knowledge gaps need to be filled for NSM FRP strengthening to be widely adopted in building applications. Therefore, the main objective of current research is to undertake comprehensive studies for tracing the fire response of RC beams strengthened with NSM FRP.



(a) Application of NSM FRP



(b) NSM FRP reinforcement

Figure 1.1 Application of NSM FRP on concrete members (For interpretation of the references to color in this and all other figures, the reader is referred to the electronic version of this dissertation)

1.2 Strengthening Strategies for Concrete Structures

In light of accelerating deterioration in civil infrastructure, a number of strengthening strategies for concrete structures have been developed since 1980s. Swamy et al. (1987) proposed a method of bonding steel plates on tension and side surfaces of beams and slabs, and the studies showed this method can increase flexure and shear strength of concrete structural members. Priestly et al. (1994) strengthened concrete columns utilizing steel jackets, and this approach was proved capable of enhancing both strength capacity (axial, flexural, and shear) and ductility of concrete columns. Some other researchers also successfully applied external post-tensioning technique to strengthen concrete beams with steel tendons (Bruggeling 1992). These techniques mainly utilize steel reinforcement plates to enhance strength capacity of concrete elements. However, the disadvantages of using steel element, including susceptibility to corrosion, difficulty to anchor, made these techniques not feasible and viable in many applications.

More recently, many of the above strengthening strategies have been tried using fiber-reinforced polymer (FRP) in place of steel. FRP is a composite which is made of high-strength fibers and a matrix for binding these fibers into structural shapes. This composite has the characteristics of high strength-to-weight ratio, good resistance to electrochemical corrosion, etc. Thus application of FRP could overcome many shortcomings of the above strengthening techniques using steel. A most common implementation of FRP strengthening is to apply FRP laminates to the surface of a concrete element, which is designated as externally bonded reinforcing (EBR) technique. In this technique, FRP sheets are saturated on site with resin, and then bonded to the

concrete with the appropriate adhesive. Some practitioners also applied pre-cured systems, where FRP sheets are saturated and cured prior to site delivery and then applied to concrete surface with adhesive. In both methods, externally bonded FRP can provide effective flexural or shear strengthening for beams, or provide seismic confinement for columns. However, research to date indicates that EBR system has a number of limitations in practice. The main limitation relates to insufficient bond between concrete and FRP sheets, which usually causes premature failure of FRP strengthening system, as shown in Figure 1.2. Consequently, design guidelines for EBR system often recommend strict strain limits for FRP reinforcement, and this leads to uneconomical use of FRP material.

In response to sensitivity of EBR system to premature debonding, researchers have proposed use of near-surface mounted (NSM) FRP reinforcement as an alternative strengthening approach. The application of this technique on an RC beam is illustrated in Figure 1.1. It can be seen that in NSM strengthening technique, the bond between FRP and concrete substrate is established on the entire surface of FRP strip or rod, and this ensures tensile or shear forces to be effectively transferred from concrete to NSM FRP reinforcement. If strengthening requires several NSM strips or rods, more parallel grooves can be cut in a specified distance and multiple FRP strips or rods can be added.

Compared to EBR technique, NSM FRP strengthening system has a number of advantages: (a) the amount of site installation work may be reduced, as surface preparation other than grooving is not required (e.g. removal of plaster and weak laitance layer is not necessary; irregularities of concrete surface is easily accommodated); (b) NSM FRP is less prone to debonding from concrete substrate, since FRP is bonded with

concrete on the entire faces of FRP strip or rod; (c) NSM FRP can be more easily anchored into adjacent members, and this feature is particularly attractive in flexural strengthening of beam-column frame, where the maximum moment typically occurs at the ends of the member; (d) NSM FRP reinforcement can more easily be pre-stressed; (e) NSM FRP reinforcement is protected by concrete cover and thus is less susceptible to accidental impact and mechanical damage, fire, and vandalism; this aspect makes this technique particularly suitable for strengthening of negative moment regions of beams/slabs; (f) the aesthetic of the strengthened structure is virtually unchanged (De Lorenzis and Teng 2007). Due to above advantages, NSM FRP strengthening technique is superior to EBR technique in many cases or can be used in combination with it.

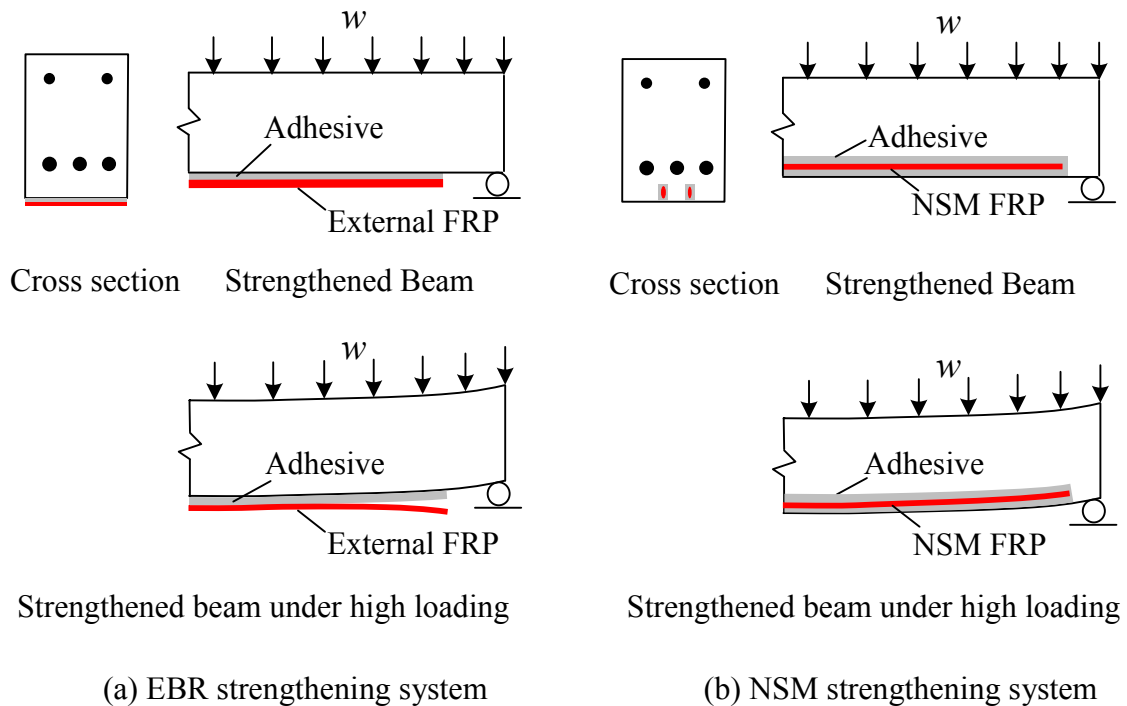


Figure 1.2 Comparison between EBR and NSM strengthening systems under bending

1.3 Behavior of FRP Strengthened RC Beams under Fire Conditions

The susceptibility of FRP to damage in fire is one of its major disadvantages. This is mainly attributed to poor performance of polymer at elevated temperatures. At ambient conditions, the molecular bonds of polymer are intact and this state is known as glassy state. As the temperature increases (about 80-150°C), the molecular bonds are weakened and a new state, leathery state, is reached. The range between glassy and leathery state is known as glass transition zone, and the corresponding temperature at which this transformation occurs is referred to as glass transition temperature (T_g) (Ashby and Jones 1999). When the temperature in FRP exceeds that of T_g , the strength and stiffness of FRP start to decrease. As the temperature further increases to 300-400°C, the molecular bonds are severely damaged and polymer matrix starts to decompose, with release of smoke, soot and toxic volatiles. These released heat, smoke and gases, during the burning of decomposition, can make fire extremely hazardous, and increase the possibility of serious injury and death. From the point view of structural behavior, FRP material may experience creep and distortion due to decomposition of polymer matrix, and this will result in significant degradation of strength and stiffness in FRP reinforcement.

When FRP reinforcement is applied in strengthening concrete structures, weak fire properties of FRP influences the fire resistance of strengthened concrete members. Typically, a conventional RC beam possesses required fire resistance for building applications, as long as appropriate concrete cover is provided to steel rebars. However, when an RC beam is strengthened with FRP reinforcement, the fire resistance of strengthened beam depends on both properties of original concrete beam and properties

of added FRP. Since FRP has much faster strength and stiffness degradation with fire exposure time than those of steel rebar, the strength capacity of FRP strengthened beam decreases faster as compared to conventional RC beam, and failure will occur when moment due to applied loading exceeds the remaining moment capacity of beam. Therefore, under the same level of loading, fire resistance of FRP strengthened RC beam is lower than that of conventional RC beam. Therefore, poor performance of FRP under fire conditions has become a key issue that hinders its use in civil infrastructures where fire safety is a concern.

Another critical issue affecting the fire response of FRP strengthened beam is bond degradation between FRP and concrete under fire conditions. In EBR strengthening system, FRP laminates are usually bonded to the external surface of concrete members through epoxy-based adhesives. These adhesives are capable of generating good adhesion at ambient conditions. However, under fire conditions, due to direct exposure to fire, epoxy-based adhesive easily gets softened and melted, and the bond strength between FRP and concrete decreases significantly. When a certain temperature (e.g. T_g for epoxy) is reached, the bond strength might be smaller than shear stress at FRP-concrete interface, leading to debonding of FRP laminates. Once debonding occurs, FRP strengthening hardly contributes to flexural or shear strength of concrete members. Based on recent experimental research performed by Firino et al. (2012), external FRP laminates debonded with original RC beam at about 20 minutes into fire, even though the anchorage zone of FRP laminates was thermally protected. Therefore, current provisions in design standards (ACI 440.2R 2008, FIB Bulletin 14 2007) do not consider the contribution of FRP strengthening under fire conditions.

In light of susceptibility of FRP and epoxy adhesive to fire damage, fire insulation has to be applied to achieve sufficient fire resistance on FRP strengthened RC beams. Previous studies show that an RC beam externally strengthened with FRP system can achieve two to four hours of fire resistance, if fire insulation is provided (Blontrock et al. 2000, Williams et al. 2008, Ahmed and Kodur 2011). However, application of fire insulation is usually expensive and time consuming, which may not be practical (due to limited space available) and economical for wide range of applications.

So far most studies on fire response of FRP strengthened RC members mainly focused on the behavior of EBR FRP strengthened concrete members. There is very limited information on NSM FRP strengthened RC members. Unlike externally bonded FRP, NSM FRP reinforcement is embedded into concrete substrate, and concrete cover provides certain level of protection to NSM FRP in the event of fire. Therefore, in the event of fire, NSM FRP experiences slower temperature rise than that of FRP in external strengthening system, so the strength degradation of FRP is alleviated. Also, high temperature resistance materials (such as cement-based material) can be applied as adhesive so that the bond between FRP and concrete might remain effective for a longer time in the event of fire. Based on recent numerical studies presented by Kodur and Yu (2013), the temperature in NSM FRP is about 300°C lower than that in external FRP at most fire exposure duration, as shown in Figure 1.3. This indicated NSM FRP retains much higher strength and stiffness than those of externally bonded FRP. Therefore, NSM FRP strengthened RC member might achieve satisfactory fire resistance for building application. However, fire response of FRP strengthened member is a complicate

problem. A comprehensive study is required to evaluate thermal and structural response of NSM FRP strengthened RC member under fire conditions.

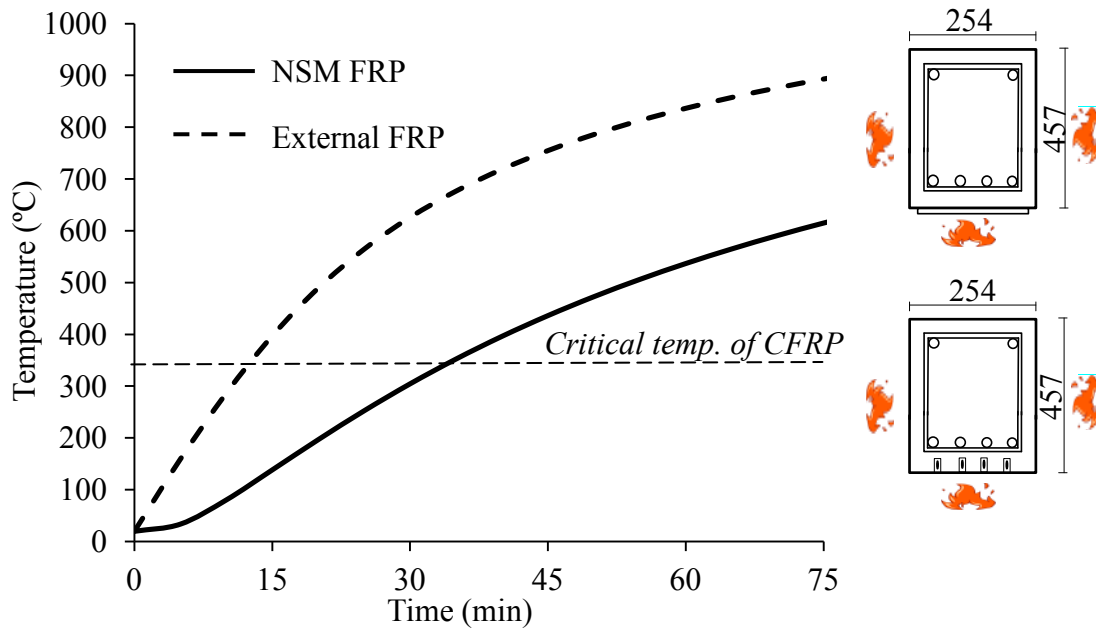


Figure 1.3 Comparison of temperature in NSM FRP and external FRP under standard fire

1.4 Objectives

From the above discussion, it is clear that there is a need for developing a comprehensive understanding on the fire response of NSM FRP strengthened RC members. To achieve this objective, both experimental and numerical studies are proposed to examine relevant high temperature material properties of NSM FRP as well as to evaluate thermal and structural behavior of NSM FRP strengthened RC beams under fire conditions. The specific research objectives of proposed study are as follows:

- Conduct a detailed state-of-the-art review on high temperature properties of FRP and on the fire response of FRP strengthened RC beams.

- Carry out high temperature property tests on NSM FRP strips and rods to evaluate the influence of high temperatures to tensile strength, bond strength, and thermal expansion properties.
- Conduct fire resistance experiments to evaluate the behavior of NSM FRP strengthened RC beams under different fire, loading, and restraint conditions.
- Develop a sophisticated macroscopic finite element based model for predicting the response of NSM FRP strengthened concrete beams under any given fire and loading conditions. Such model will account for nonlinear high temperature properties of constituent materials, various strain components, fire induced restraint effects, as well as temperature induced bond degradation at FRP-concrete interface.
- Validate the above numerical model by comparing predicted parameter response against data from fire resistance tests.
- Carry out parametric studies to quantify the influence of various factors on the fire resistance of FRP reinforced or strengthened concrete beams.
- Utilizing results obtained from experimental and numerical study, develop simplified rational design methodology for evaluating fire resistance of FRP strengthened concrete beams.

1.5 Scope

The work presented in this dissertation involves experimental and numerical studies on characterization of fire performance of NSM FRP at both material and structural levels. As part of experimental research, extensive high temperature property tests on NSM FRP strips and rods were undertaken for characterizing mechanical, bond, and deformation

properties of NSM FRP reinforcement at material level. At structural level, four full scaled RC T-beams strengthened with NSM FRP were fabricated and tested under standard and design fire conditions, to evaluate the fire response of NSM FRP strengthened RC members. To develop further understanding on critical factors influencing fire resistance of NSM FRP strengthened RC beam, a macroscopic finite element model available in literature was extended to trace the response of NSM FRP strengthened RC beam from pre-loading stage to collapse. Data from fire resistance tests was utilized to validate the macroscopic finite element model. The validated numerical model was then applied to carry out parametric studies to quantify influence of various factors on fire response of NSM FRP strengthened RC beam. The dissertation is organized into eight chapters as follows:

- Chapter 1 provides background information on strengthening of beams through NSM technique, and lays out objectives of the dissertation.
- Chapter 2 provides a literature review on room temperature behavior of NSM FRP strengthening, and also summarizes high temperature material properties of concrete, steel and FRP. A review of recent experimental and analytical studies on fire response of RC beams incorporated with FRP reinforcement is also provided.
- Chapter 3 presents high temperature property tests on tensile strength and modulus, bond strength, and thermal expansion of NSM FRP. Empirical relationships for predicting high temperature properties of NSM FRP are developed over a wide temperature range.

- In Chapter 4, details on fire resistance experiments on NSM FRP strengthened T-beams are presented. Results from the fire tests are utilized to discuss the comparative response of NSM FRP strengthened RC beams under fire conditions.
- Chapter 5 covers details on macroscopic finite element model and analysis for predicting fire resistance of NSM FRP strengthened RC beams. Development of subroutines based on high temperature properties of NSM FRP, as well as validation of the extended numerical model, are also presented in this chapter.
- Chapter 6 presents results from parametric studies to illustrate the influence of critical factors on fire response of NSM FRP strengthened RC beams.
- Chapter 7 provides a rational design methodology developed based on experimental and numerical studies. Such methodology can be applied to predict the fire response of FRP strengthened RC beams under different scenarios.
- Chapter 8 summarizes the key findings, recommendations for future work and research impact based on this study.

CHAPTER 2

STATE-OF-THE-ART REVIEW

2.1 General

The use of FRP composites in aerospace and automotive industry has started since 1950s, due to their superior properties such as high strength to weight ratio and excellent resistance to corrosion. Starting from 1990s, with decreasing cost of FRP products, fiber-reinforced polymer (FRP) has been increasingly used in civil engineering applications, especially as strengthening and retrofitting for concrete structures. Multiple advantages of FRP strengthening have been demonstrated in civil constructions, such as ease of application, cost effectiveness, as well as efficient performance. Therefore, wide varieties of structural elements are being strengthened using FRP including beams, slabs, columns, and shear walls.

Through extensive studies and applications in last two decades, it is believed that fire behavior is an important factor limiting the wider use of FRP in many areas (Mouritz and Gibson 2006). This is mainly attributed to faster strength and stiffness degradation of FRP under fire conditions. Therefore, there is a concern on application of FRP strengthening in building or other places where the fire performance of structural members is a major design requirement.

This section provides a state-of-the-art review on fire performance of FRP as material and as structural system. The review starts with an introduction of NSM FRP technique and its application in concrete structural members, followed by a review of

high temperature properties of constituent materials of FRP strengthened RC member (concrete, reinforcing steel, FRP and insulation). Then the main findings from previous experimental and numerical studies on fire response of concrete beams incorporated with FRP reinforcement are discussed, including concrete beams reinforced with internal FRP rebars, RC beams strengthened with external FRP laminates, and RC beams strengthened with NSM FRP. Finally, design provisions in current codes and standards for FRP strengthened structural members are reviewed.

2.2 Configuration and Installation of NSM FRP Strengthening

The application of NSM steel rebars in Europe for the strengthening of RC structures dates back to the early 1950s (Asplund 1949). In 1948, an RC bridge in Sweden experienced excessive settlement of the negative moment reinforcement during construction, and thus the negative moment capacity needed to be increased. The strengthening was accomplished by grooving the surface, filling the grooves with cement mortar and embedding steel rebars in the grooves. The arrival of FRP as NSM reinforcement has amplified the advantages of NSM technique. As comparing to steel, FRP reinforcement possesses better resistance to corrosion, facilitates installation and construction due to its lightweight, and reduces the size of the groove due to its higher tensile strength. This section specifically presents the materials and installation of NSM FRP strengthening.

2.2.1 NSM FRP reinforcement and groove filler

In current NSM FRP applications, carbon FRP (CFRP) reinforcement has been mostly used to strengthen concrete structures. Glass FRP (GFRP) has been used in many NSM applications in masonry and timber structures. The tensile strength and elastic modulus of CFRP are much higher than those of GFRP. Thus, for the same tensile capacity, CFRP reinforcement has a smaller cross-sectional area than that of GFRP reinforcement, and a smaller groove is needed, which leads to easier installation, less risk of interfering with the internal steel reinforcement, and also savings in the groove-filling material. Round (rod) and rectangular (strip) FRP bars are popular shapes used in NSM applications. The rods are of 6 or 10 mm in diameter manufactured as a deformed reinforcement. While NSM strips have a rectangular cross section, with typical dimensions of 2-5 mm in thickness and 16 mm in width (See Figure 1.1b). The rods are usually delivered to the site and cut to the required length, while the NSM strips are delivered in rolls no greater than 250 feet in length (Hughes Brothers 2011). Each cross-sectional shape has its own advantages. For example, narrow strips maximize the surface area-to-sectional area ratio for a given volume and thus minimize the risk of debonding, while round bars are more readily available and can be more easily anchored in pre-stressing operations. In practical applications, the choice depends strongly on the depth of the cover, and the availability and cost of a particular type of FRP bar (De Lorenzis and Teng 2007).

Groove filler is the medium for transferring stresses between FRP bar and concrete. In terms of structural behavior, the most relevant mechanical properties of groove filler are tensile and shear strengths, since the bond capacity of NSM reinforcement is controlled by cohesive shear failure of the groove filler. Based on

published test data (De Lorenzis et al. 2002, Al-Mahmoud et al. 2011, Burke et al. 2012, De Lorenzis and Teng 2007), the most common and best performing groove filler is a two-component epoxy. The two components, resin and hardener, need to be thoroughly blended using a mixer before filling into the groove. Usually the epoxy is designed with a high-viscosity material to avoid dripping or flowing-away, and to accelerate the hardening of material. Well-hardened epoxy is characterized by having excellent weathering resistance and good temperature resistance. The use of cement paste or mortar as a groove filler is also explored in an attempt to lower material cost, reduce hazard to workers, and achieve better resistance to high temperatures. However, cement mortar has inferior mechanical properties and durability, with a tensile strength an order of magnitude smaller than that of common epoxies (Taljsten et al. 2003, Burke et al. 2013).

2.2.2 Installation procedure

Compared to externally bonded FRP technique, installation of NSM FRP for strengthening has a relatively simple procedure, and requires less skill during the installation. Based on experience gained during the laboratory installation process, a recommended field application procedure for NSM FRP has been developed as follows (Hughes Brothers 2011).

- Step 1: Grooves are cut after making the layout on the surface of concrete member. Proper equipment such as diamond crack chasing blades, guide rails and sufficiently sized power tools can make groove cutting easier. Rather than cut the groove in a single pass, sometimes it is more efficient to cut parallel grooves and remove the concrete between the saw cuts.

- Step 2: Chisel any remaining concrete between cut paths.
- Step 3: Clean the groove and eliminate any residual dust with compressed air or vacuum.
- Step 4: For a clean appearance, mask the concrete adjacent to the groove.
- Step 5: Fill the groove approximately half way with adhesive.
- Step 6: Center and insert FRP strip or rod in the groove. The strip or rod should be inserted until approximately flush with the surface of the concrete and shall be approximately centered in the groove before final seating.
- Step 7: Fill the entire groove with epoxy. Hardened epoxy should not extend more than $\frac{1}{4}$ inch from the edge of the groove.
- Step 8: Cure epoxy until full recommended time limit before resuming traffic flow. The recommended time limit should be based on conditions during application.



(a) Cut grooves on concrete cover



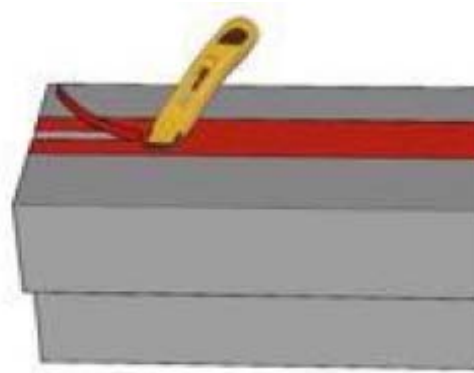
(b) Chisel remaining concrete in groove

Figure 2.1 Procedures of installing NSM FRP (Hughes Brothers 2011)

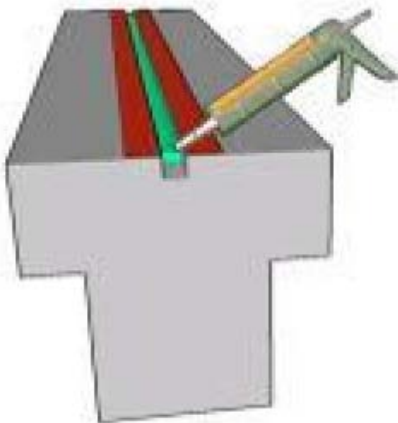
Figure 2.1 (cont'd)



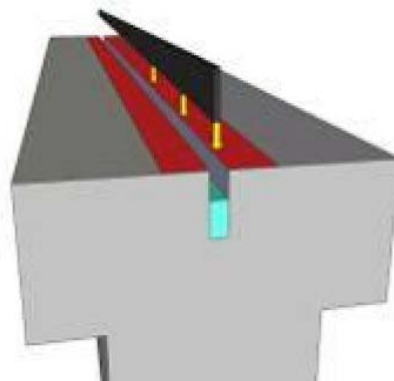
(c) Clean grooves eliminate residual dust



(d) Mask concrete close to groove



(e) Fill groove half way with adhesive



(f) Center and insert NSM FRP strip

There are a few recommendations which should be followed during the installation of NSM FRP, to ease the application process and to achieve better bonding effect. Firstly, the location and dimension of NSM groove should be appropriately designed. Based on the recommendation from ACI 440.2R specifications (2008), the minimum dimension of the grooves for NSM strengthening should be taken at least 1.5 times the diameter of FRP bar. When a rectangular bar with large aspect ratio is used,

however, the limit may lose significance due to constructability. In such a case, a minimum groove size of $3.0a_b \times 1.5b_b$, as depicted in Figure 2.2, is suggested, where a_b is the smallest bar dimension. The minimum clear groove spacing for NSM FRP bars should be greater than twice the depth of the NSM groove to avoid overlapping of the tensile stresses around NSM bars. Furthermore, a clear edge distance of four times the depth of NSM groove is recommended to minimize the edge effects that could accelerate debonding failure (Hassan and Rizkalla 2003).

Secondly, proper equipment is important to maintain uniform depth and thickness during cutting. Theoretically, any configuration of blades is allowable, as long as the minimum required thickness and depth can be achieved. When choosing a saw for cutting the grooves, three viable options exist: a track mounted saw, a hand saw, and a standard joint-cutting saw. Equipment availability and cost efficiency will determine the best method for cutting.

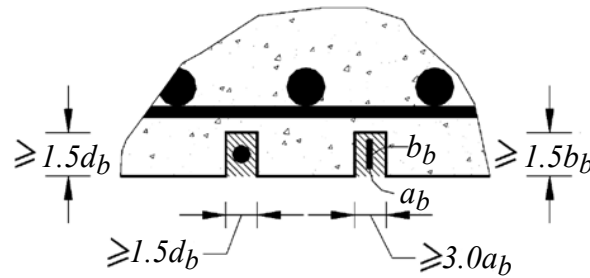


Figure 2.2 Requirement on dimensions of NSM groove (ACI 440.2R 2008)

Thirdly, during the consolidating of epoxy, any tools with a small profile, roughly one-quarter the width of the groove, can be applied to eliminate air voids created during the injection process. Likewise, a few spacers with any approximate thickness of 1/16

inch can be employed to center FRP strip or rod in the groove during the epoxy filling. This also ensures a proper bond generated on all the faces of NSM FRP rebar.

In field application of NSM FRP strengthening, the sequence of the above procedure may be slightly different, depending on timeline of availability of FRP rebar and adhesive products, cutting tools, etc. However, as long as NSM FRP is inserted at proper position of FRP and sufficient bond is generated between FRP and concrete substrate, the installation of NSM FRP strengthening is considered to be successful.

2.3 Behavior of NSM FRP Strengthened Members at Ambient Conditions

In recent years, near-surface mounted (NSM) FRP technique has received a great deal of attention in civil engineering community. Considerable research has been conducted on the behavior of NSM FRP bond and NSM FRP strengthened structural members at ambient conditions. This section provides an overall review of previous studies on the behavior of NSM FRP strengthening at room temperatures.

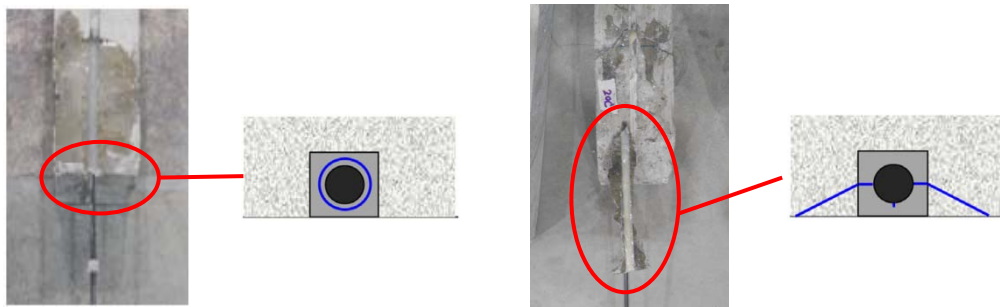
2.3.1 Bond behavior of NSM FRP system

In NSM FRP strengthened concrete members, NSM FRP reinforcement and concrete substrate are bonded together through epoxy or cementitious adhesives, the assemblies of NSM FRP, adhesive and concrete substrate can be referred to as NSM FRP system. It is no doubt that bond properties of NSM FRP system play a critical role in ensuring the effectiveness of FRP strengthening.

A review of literature shows that a number of studies have been carried out on bond properties of NSM FRP system at ambient conditions. Results from these studies

indicate that bond strength and modulus of NSM FRP system at ambient conditions depends on a number of parameters such as FRP type, cross-sectional shape adhesive materials, concrete strength, etc. These parameters can be grouped under two primary factors, namely, roughness of contact surfaces (FRP or concrete surface) and shear strength of groove adhesive. These two factors influence the mode of failure at concrete-epoxy-FRP interface, and thus can produce varying bond strengths.

For an FRP strip or rod with a smooth or lightly sand-blasted surface, bond failure usually occurs at FRP-epoxy interface, either through pure interfacial failure or cohesive shear failure in the groove filler (De Lorenzis et al. 2002, Teng et al. 2006, Al-Mahmoud et al 2011). However, for an FRP strip or rod with large deformation or sand coating on the surface, NSM epoxy develops strong adhesion with FRP rebars, and thus bond failure mostly occurs at epoxy-concrete interface, through fracture at concrete edge or cracking of epoxy (Sena Cruz and Barros 2004, De Lornezis and Nanni 2002, Bilotta et al. 2011). Failure at epoxy-concrete interface usually produces higher bond strength than that at FRP-epoxy interface. The illustration of these two failure modes is plotted in Figure 2.3.



(a) Failure at FRP-epoxy interface

(b) Failure at epoxy-concrete interface

Figure 2.3 Typical failure modes of NSM FRP system

Other than failure modes of debonding, local bond-slip behavior is another important aspect for evaluating the bond behavior of NSM FRP systems. Typically the

local bond stress-slip response can be grouped under two distinct stages: pre-peak stage and post-peak stage, as shown in Figure 2.4. In pre-peak stage, the bond stress increases at a high rate and quickly reaches its peak value, and the slip between FRP and concrete is quite small. In this stage, there is good adhesion between FRP and adhesive, and the measured slip is roughly equivalent to elastic deformation of CFRP and adhesive. Past the peak point (post-peak stage), the bond stress drops quickly, and this is mainly due to damage or deterioration of adhesive. In this stage, NSM system might drop abruptly to a very small value (close to zero), or decrease gradually until the FRP is pulled out, depending on failure modes of NSM systems (De Lorenzis et al. 2004, Sena Cruz and Barros 2004). In previous NSM bond test, the local bond-slip behavior of NSM strips from two different tests are very close to each other and are comparable to that of spirally wound bars (Sena Cruz and Barros 2002, Blaschko et al. 2003). Thus some mathematical models were proposed to predict the local bond-slip behavior of NSM FRP (De Lorenzis et al. 2004, Sena Cruz and Barros 2004). However, due to numerous variations in FRP reinforcement and adhesive materials, a variety of factors can influence the failure of NSM bond. Thus further studies are still needed to develop complete understanding on debonding mechanism of NSM FRP system at room temperature.

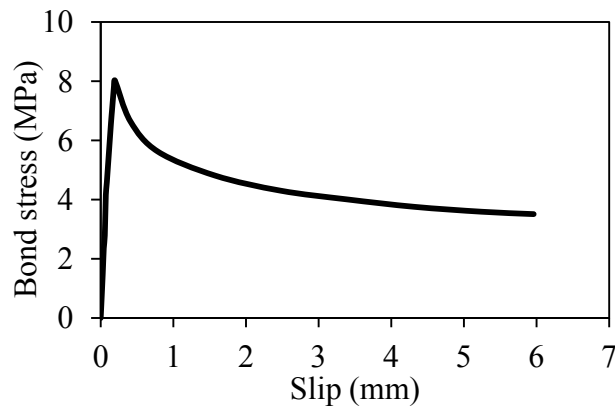


Figure 2.4 Typical bond-slip curve of NSM FRP system

2.3.2 Behavior of NSM FRP strengthened RC members

Results from existing studies on strengthened beams, slabs, and columns indicate that provision of NSM FRP reinforcement enhances their flexural capacity, both at yielding of steel reinforcement and ultimate conditions, and post-cracking stiffness. Some test programs compared the performance of EBR with NSM systems, by strengthening identical beams with equivalent amounts of FRP. In all cases, NSM FRP achieved a higher strain during debonding or no debonding occurred (El-Hacha and Rizkalla 2004, Alkhrdaji et al. 1999, Hassan and Rizkalla 2002). Thus NSM FRP reinforcement performed more effectively as compared to externally bonded FRP. El-Hacha and Rizkalla (2004) also compared equivalent amounts of NSM reinforcement provided with round bars or strips. As expected, NSM strips performed better, and failed by tensile rupture as compared to debonding of NSM rods. This mainly results from larger lateral surface to cross-sectional area ratio of NSM strips and relatively higher local bond strength.

Based on previous experimental studies, the failure modes of NSM FRP strengthened RC beams can be categorized in two main types. One possibility is composite action between the original beam and NSM FRP is well maintained until the failure of beam. In these beams, the failure occurs through crushing of top concrete or rupture of FRP, after the yielding of internal steel bars. Another failure mode is the “premature” debonding failure of NSM FRP system, which involves the loss of composite action at FRP-concrete interface (De Lorenzis and Teng 2007). The debonding mode of NSM FRP on flexural members depends on several parameters, including internal steel reinforcement ratio, FRP reinforcement ratio, cross-sectional shape and

surface configuration of NSM reinforcement, and tensile strength of epoxy and concrete. So far there is still limited understanding of the mechanism of debonding in beams strengthened with NSM FRP system. Descriptions of failure modes in the existing literature are often not sufficiently detailed to understand the progression of failure process. Thus the design guidelines (ACI 440.2R 2008) recommended a reduction factor (0.7) in the ultimate strain of NSM FRP to account for the uncertain debonding failure.

Another important issue in the design of NSM FRP strengthened RC beam is the prediction of flexural strength. If the failure of a strengthened beam does not occur through debonding, then the ultimate load capacity at which failure occurs can be easily predicted using equations developed for externally bonded FRP based on the plane section assumption (Teng et al. 2002). While accurate prediction on failure loads at debonding is much more challenging. Some researchers have proposed theoretical models to evaluate the ultimate load capacity of NSM FRP strengthened beams (Teng et al. 2003, Lu et al. 2007), but these models only have limited use for certain types of NSM FRP system, or certain debonding failure modes. Further research is needed to acquire a thorough understanding of mechanics of composite action between NSM FRP and concrete substrate. Then a more sophisticated model can be developed for tracing the response of flexural members strengthened with NSM FRP reinforcement.

2.4 Material Properties at Elevated Temperatures

The fire response of concrete members incorporated with FRP reinforcement is influenced by high temperature properties of constituent materials. Specifically, these properties include thermal, mechanical and deformation properties. The thermal

properties govern the extent of heat transfer within structural members, while mechanical properties influence the load carrying capacity and deformation of structural member. The deformation properties, mainly referring to thermal expansion and creep, determine the extent of deformation of structural member under certain loading. This section provides a review on properties of concrete, reinforcing steel, FRP reinforcement and insulation materials which are typically used in building construction.

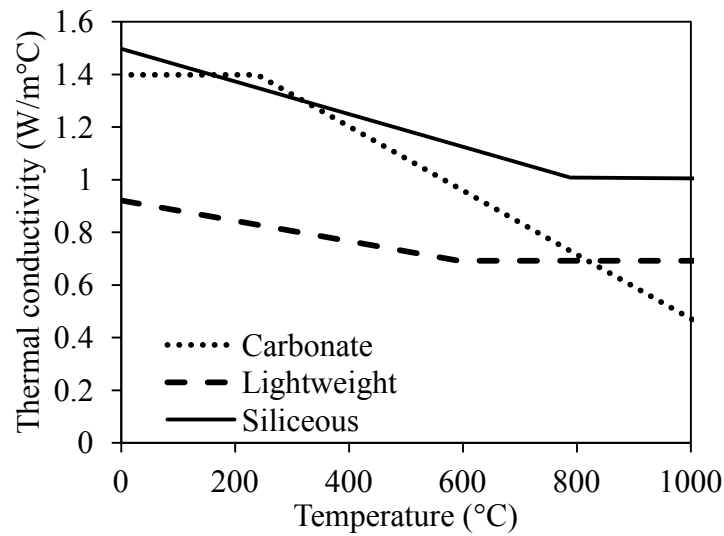
2.4.1 Concrete

Concrete has been used as construction material for hundreds of years. The information on variation of thermal properties of concrete with temperatures is well established, based on extensive experimental and theoretical studies. Since normal strength concrete is usually used in FRP strengthened concrete members, the literature review herein mainly focuses on the properties of this type of concrete.

2.4.1.1 Thermal properties

Thermal properties of concrete, which mainly refer to thermal conductivity, specific heat and density, have dominant influence on thermal response of concrete members under fire conditions. A great deal of research has been conducted on variation of thermal properties of concrete at elevated temperatures, and there are also some recommendations on temperature-properties relations in various codes and standards. Three major types of concrete are commonly used in buildings, namely, siliceous concrete, carbonate concrete, and lightweight concrete, which are categories based on the type of aggregate. Figure 2.5 illustrates the variation of thermal properties of different

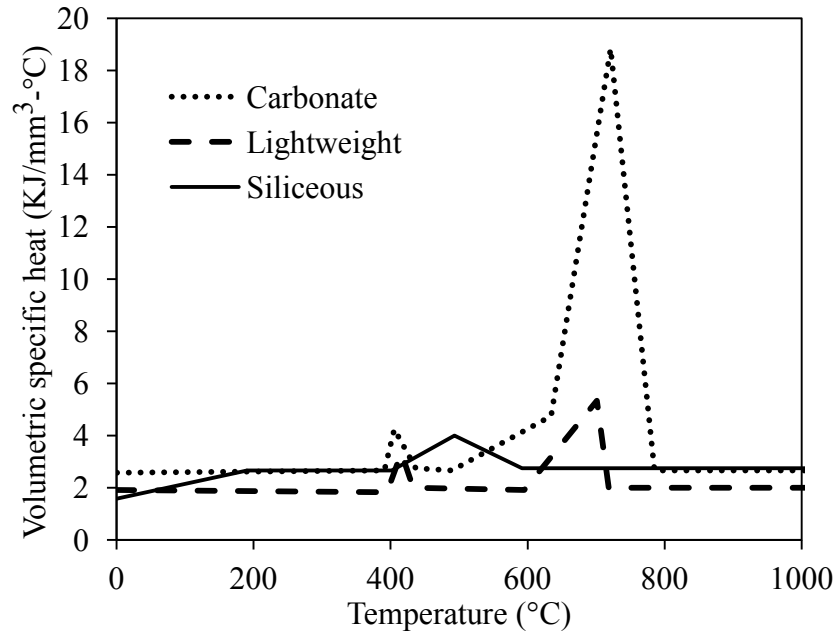
concrete as a function of temperature (Lie 1992, Kodur et al. 2008). In Figure 2.5(a), it can be seen that the thermal conductivity of carbonate concrete tends to decrease with increased temperature. Comparably, siliceous concrete has a relatively larger initial value of thermal conductivity, but it decreased more rapidly with temperatures. Lightweight concrete, on the other hand, shows nearly constant thermal conductivity over a wide range of temperature. Thus, thermal properties of concrete vary significantly depending on types of aggregate used in the batch mix.



(a) Variation of thermal conductivity for various types of concrete (Lie 1992)

Figure 2.5 Variation of thermal properties with temperature for various types of concrete

Figure 2.5 (cont'd)



(b) Variation of specific heat with temperature for various types of concrete (Lie 1992)

The specific heat of different concrete is presented in Figure 2.5(b) (Lie 1992). Overall, the specific heat values of three types of concrete are close, except that of carbonate concrete which has much higher values at around 700°C. The character of cement paste and aggregate contributes to these distinct peaks. It can be found that carbonate aggregate concrete possess a higher specific heat and lower thermal conductivity, as compared to siliceous concrete. Thus carbonate concrete is usually preferred over siliceous aggregate, when a superior high temperature behavior is required in structural members (Kodur et al. 2008).

Some studies indicated thermal conductivity and specific heat of concrete also depend on moisture content and concrete porosity (Naus 2006, Flynn 1999). Therefore, in the structural fire design guidelines in Eurocode 2 (2004), the influence of moisture content is incorporated in the variation of specific heat as a function of temperatures.

2.4.1.2 Mechanical properties

Two types of studies have been conducted on the variation of mechanical properties of concrete with temperature. One is to measure the properties during exposure to certain high temperatures, and this measurement can be used to simulate the behavior of concrete members during heating phase of fire (Lie and Kodur 1996, Khoury 1996, Cheng et al. 2005). Another type of study is to evaluate mechanical properties after exposure to high temperatures, and these measured values are mainly used to simulate the behavior of concrete members during cooling phase of fire or post-fire behaviors (Lau and Anson 2006, Chang et al. 2006, Savva et al. 2005). In this section, a review of mechanical properties of concrete at both heating and post-heating phases is provided, mainly including the variation of compressive strength and elastic modulus with temperature.

The variation of compressive strength of concrete at heating phase is plotted in Figure 2.6 (Kodur et al. 2008). It can be seen for different types of concrete, the compressive strength follows a similar degradation trend as a function of temperature. In 20-300°C temperature range, no strength degradation is observed for all types of concrete. Beyond 400°C, concrete strength decreases quickly, due to changes developed in the internal concrete structures. It can be noticed that there is a clear difference on the strength degradation at elevated temperature in ASCE and Eurocode 2 models. A major reason for this difference is the ASCE manual (Lie 1992) does not specifically account for the effect of aggregate types on compressive strength of concrete at elevated temperatures. It can be seen that ASCE model is roughly the upper bound of test data,

while Eurocode model is close to the lower bound. Based on the results of recent numerical studies (Kodur et al. 2008), both ASCE manual and Eurocode give conservative predictions on fire resistance of columns made of carbonate concrete. However, ASCE constitutive model provides better predictions in the simulations as compared to Eurocode constitutive model.

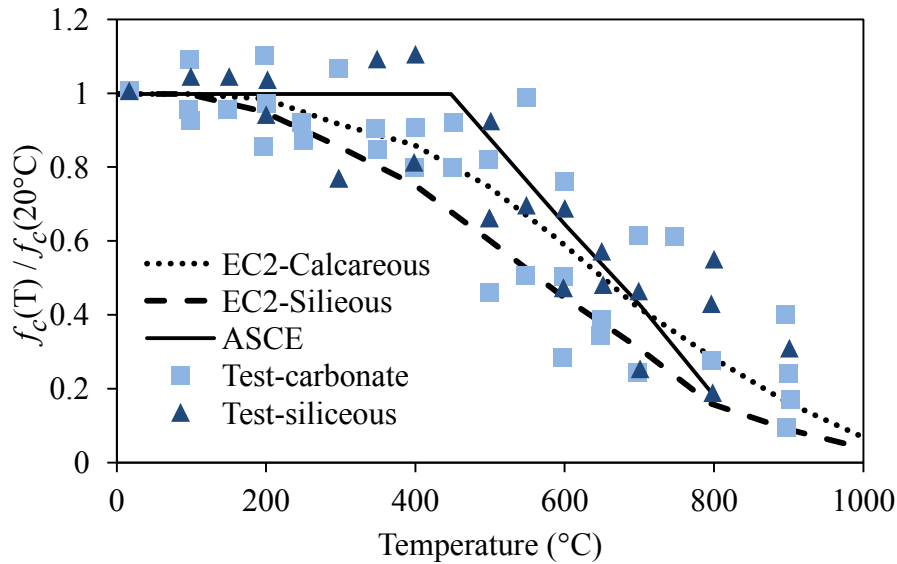


Figure 2.6 Variation of compressive strength with temperature for various types of concrete (Kodur et al. 2008)

The variation of elastic modulus with temperature for different concrete aggregate is shown in Figure 2.7 (Schneider 1988). It can be seen that the modulus of elasticity of concrete decreases starting from room temperature, which is different from the degradation trend of compressive strength of concrete. At 400°C, only 40-50% of the original modulus of elasticity is retained for siliceous and carbonate concrete. Carbonate concrete retains slightly higher modulus than that of siliceous concrete, and this can result from better temperature resistance of carbonate aggregate. Lightweight concrete has

relatively slower degradation on modulus of elasticity, and this is probably attributed to less aggregate and less voids inside of concrete.

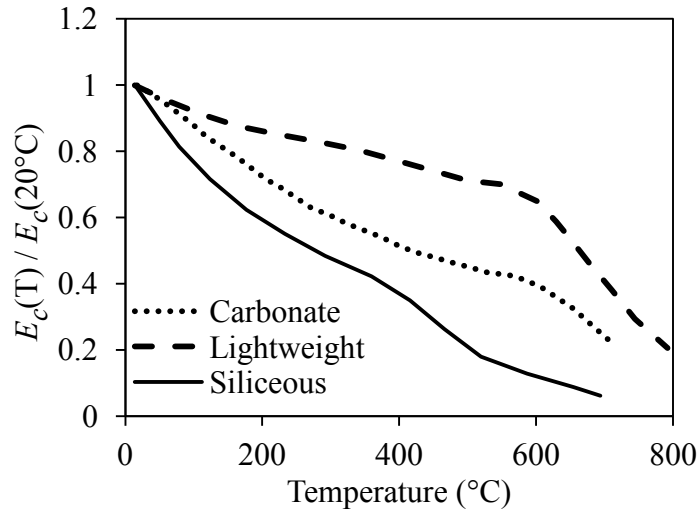


Figure 2.7 Variation of elastic modulus with temperature for various types of concrete

Mechanical properties of concrete at post-heating phase mainly refers to residual strength of concrete after heating, which is an important parameter for modeling concrete structural members exposed to design fire (the fire with cooling phase). However, both Eurocode 2 (2004) and ASCE manual (1992), do not specify any relationships for residual strength of concrete after fire exposure. Some published data on residual strength of concrete is shown in Figure 2.8 (Kumar 2003). Compared to trends in Figure 2.6, it can be seen that residual strength of concrete at a given temperature is less than that of concrete during heating. This is because during cooling phase of design fire, the process of hydration in cement components is an ongoing process. These hydrated products have larger volume that introduces more cracking in concrete, and thus concrete continues to lose strength and stiffness (Kodur and Dwaikat 2008). It can be seen that there is relatively large difference on test data of residual strength, and this can be attributed to

different heating and cooling rate during each test. The best fit of test data that can be used for evaluating the residual strength of concrete is shown in Figure 2.8 (Kumar 2003).

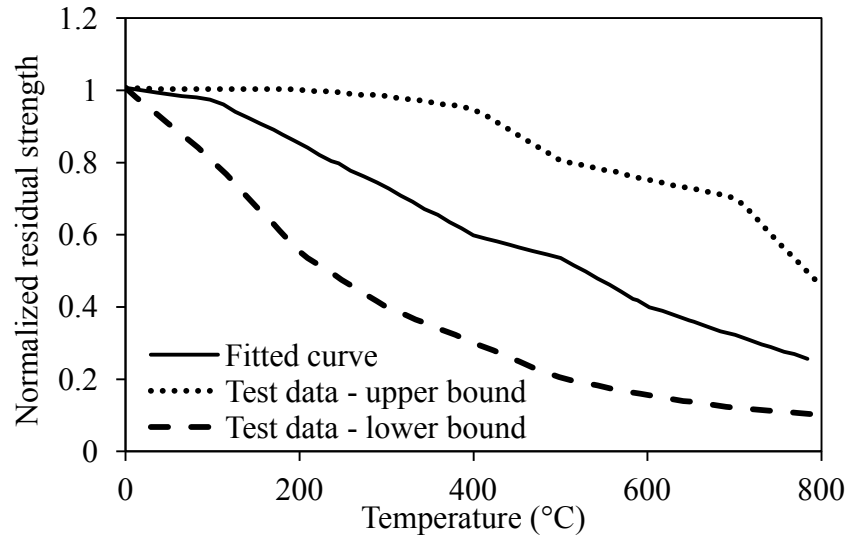


Figure 2.8 Variation of residual strength of concrete with temperature

2.4.1.3 Deformation properties

Recent research results indicate that at extreme temperatures, deformation properties, which mainly refer to thermal expansion, creep and transient strain, have important effects on strength and deformation of concrete structural members (Kodur and Dwaikat 2008). Figure 2.9 illustrates the variation of thermal strain at elevated temperatures (Lie 1992, Eurocode 2004). It can be seen that thermal expansion highly depends on the aggregate of concrete, and this is mainly attributed to the fact that coarse aggregate, which determines the extent of thermal expansion, makes up to 70-80% of total solid concrete volume. Typically, thermal expansion of concrete with siliceous aggregate is more significant as compared to concrete with carbonate aggregate. However, if concrete is subjected to stress levels larger than 35% of its ultimate strength, thermal

expansion is essentially eliminated, as it is counteracted by the applied stress (Williams 2007).

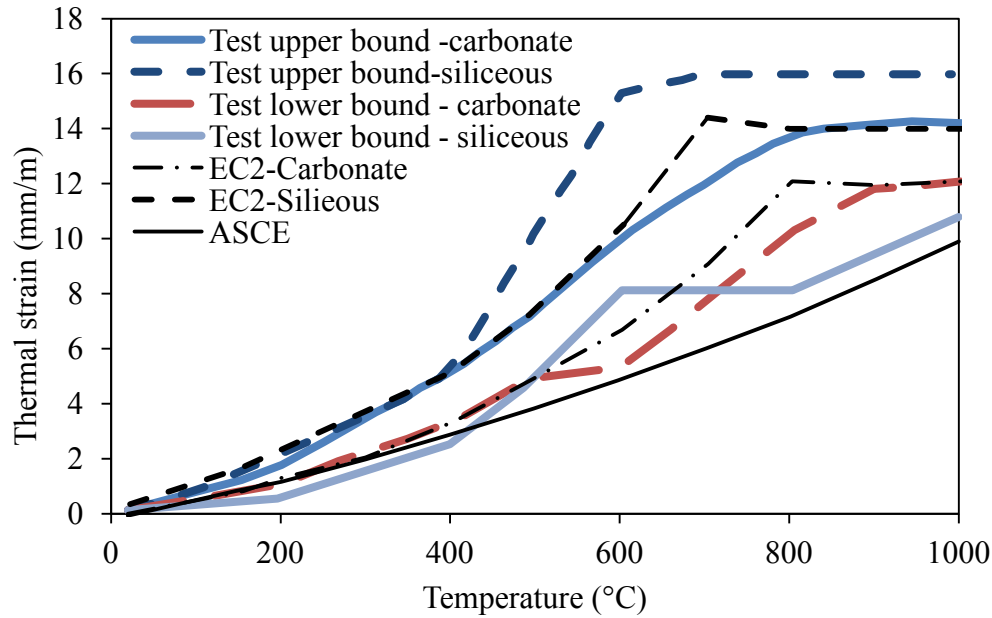


Figure 2.9 Variation of thermal strain with temperature for various types of concrete

The creep behavior of concrete is a complex problem, especially at high temperatures. At fire conditions, creep strain becomes significant since moisture movement occurs more rapidly. Creep strain depends on many factors including temperature, stress level, time, loading and mix design of concrete. Previous studies show that creep strain is significant in low-modulus aggregates, and is more pronounced at higher load level and elevated temperatures (Dwaikat 2009).

Transient strain is a phenomenon that is related to creep behavior, which develops in addition to creep during the first heating under load and is independent of time (Khouri 2000). The mismatch in thermal expansion between aggregate and cement paste leads to development of internal stresses and micro-cracking, and this results in the growth of transient strain in concrete (Schneider 1988).

There is very limited information in the literature on high temperature creep and transient strains (Kodur and Harmathy 2008). Anderberg and Thelandersson (1976) proposed an evaluation equation for creep strain of concrete at high temperature, which is

$$\varepsilon_{cr} = \beta_1 \frac{\sigma}{f_{c,T}} \sqrt{t} e^{d(T-293)} \quad (2.1)$$

where ε_{cr} = creep strain, $\beta_1 = 6.28 \times 10^{-6} s^{-0.5}$, $d = 2.658 \times 10^{-3} K^{-1}$, T = concrete temperature (K) at time t (s), $f_{c,T}$ = concrete strength at temperature T , and σ = stress in the concrete at time t (s).

Harmathy (1993) proposed a formula to predict the transient strain at elevated temperature, as shown below

$$\varepsilon_{tr} = k_2 \frac{\sigma}{f_{c,20}} \varepsilon_{th} \quad (2.2)$$

where ε_{tr} = transient strain, σ = stress in the concrete, k_2 = a constant ranges between 1.8 and 2.35, ε_{th} = thermal strain, and $f_{c,20}$ = concrete strength at room temperature.

Based on previous studies (Kodur and Dwaikat 2008, Kodur and Ahmed 2010), these equations generally produce reasonable estimates for creep and transient strains in concrete under fire conditions.

Relations for the variation of thermal, mechanical and deformation properties of concrete are given in codes and standards (Lie 1992, Eurocode 2 2004), and these are included in Appendix A.

2.4.1.4 Fire induced spalling

Fire induced spalling has received a great deal of attention in recent years. Many studies (Phan 1996, Kodur and Dwaikat 2008, Raut and Kodur 2011) have indicated the spalling can accelerate the deterioration of concrete members under fire condition, and the influence of spalling needs to be accounted in fire performance evaluations. Spalling occurs when pore pressure in concrete exceeds tensile strength of concrete, causing concrete chunks to fall off from concrete member. This falling off can often be explosive due to high pore pressure, generated from high thermal gradients.

The extent of spalling in concrete depends on many factors, and the primary factors influencing fire induced spalling are moisture content, concrete permeability, concrete strength, fire scenario, and stress level (Phan 1996, Phan et al. 2000, Kodur and Phan 2007). Compared to normal strength concrete, high strength concrete is believed more susceptible to have spalling under fire conditions. One reason might be the low permeability and high density of high strength concrete, which prevent water vapor from escaping and lead to high pore pressure that causes spalling. Also, high strength concrete is normally subjected to higher stress levels than normal strength concrete and this may increase the chances of occurrence of fire induced spalling.

Fire induced spalling could cause reduction of concrete cross-section and accelerate strength loss, and further leads to decrease in fire resistance of a concrete member. However, FRP strengthening is mainly applied to concrete members with normal strength concrete. Also, due to long term aging and deterioration, concrete in the strengthened members is in relatively low strength. Therefore, few data has been reported on the occurrence of spalling in FRP strengthened RC members, especially when beams

are protected with insulation. Thus fire-induced spalling is not a primary concern in this study.

2.4.2 Reinforcing steel

Although steel reinforcement forms only a small portion of cross sectional area in concrete members, high temperature properties of steel reinforcement, especially mechanical properties, has significant influence on the fire response of reinforced concrete members. This section reviews some notable studies on the behavior of reinforcing steel at elevated temperatures.

2.4.2.1 Thermal properties

Thermal properties of reinforcing steel mainly depend on the type of steel and temperatures in steel reinforcement. These properties include thermal conductivity and thermal capacity. It is well known that steel is a good heat conductor and its thermal conductivity is quite high as compared to other construction materials. Figure 2.10 presents the idealized values of thermal conductivity of steel reinforcement at elevated temperatures (Lie 1992). It can be seen that thermal conductivity of steel decreases linearly with increasing temperature until reaching 900°C, and then remain almost constant at higher temperatures.

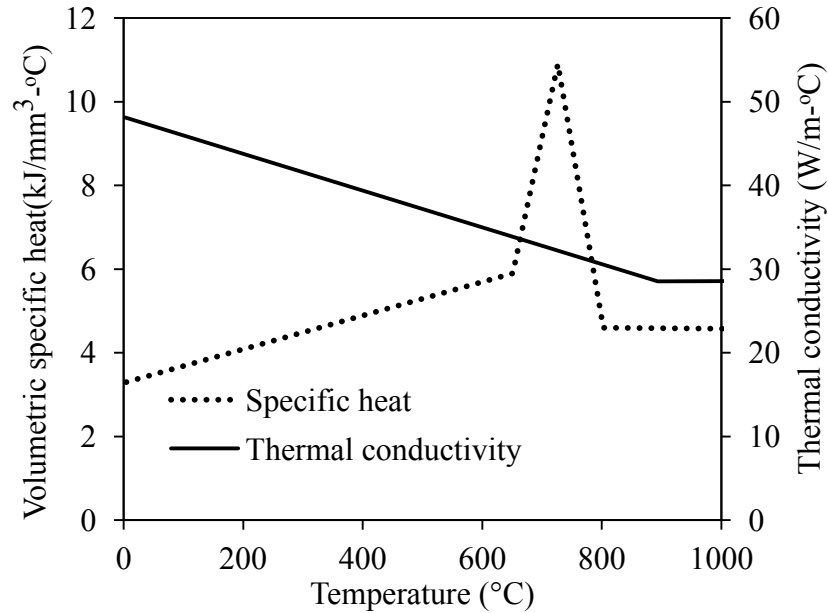


Figure 2.10 Variation of thermal properties with temperature for reinforcing steel

Specific heat, is defined as the amount of heat required to raise a unit degree of temperature in a unit volume. The variation of specific heat as a function of temperature is shown in Figure 2.10. The specific heat of reinforcing steel increases slightly at elevated temperatures, and the peak value at around 700°C can be attributed to phase transformation of steel material. As mentioned earlier, the area of steel reinforcement is much smaller as compared to the care of overall concrete, and thus thermal properties of reinforcing steel has negligible influence on temperature distribution within concrete cross section (Lie and Irwin 1993).

2.4.2.2 Mechanical properties

Since steel reinforcement primarily contributes to tensile force in an RC beam, the degradation on mechanical properties of steel reinforcement has critical influence on fire response of RC beams. Overall high temperature degradation of mechanical properties of

steel can be very different depending on the composition and strength of steel reinforcement. Figure 2.11 plots a variation of strength properties of reinforcing steel as a function of temperatures, based on the specifications as per ASCE (Lie 1992) and Eurocode 2 (2004). For yield strength, Eurocode 2 assumes that reinforcing steel retains its original strength up to 400°C, while in ASCE manual (Lie 1992) the yield strength gradually decreases starting from the initial increase in temperature. Also, Eurocode 2 does not consider strain hardening effect in steel rebar, and specifies ultimate strength is the same with yield strength. ASCE manual accounts for strain hardening after steel yields, and it specifies that degradation of ultimate strength is always slightly smaller than that of yielding strength, as plotted in Figure 2.11.

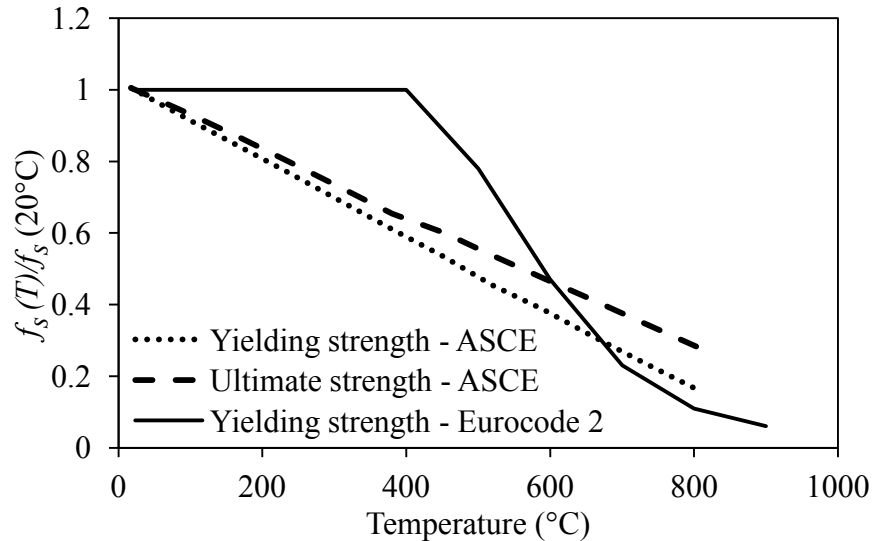


Figure 2.11 Variation of yield strength and ultimate strength with temperature for reinforcing steel

Another mechanical property of steel rebar is that original yield strength of heated steel rebar can be recovered after the cooling. Previous study shows that the yield strength of steel after cooling is almost same with the room temperature yield strength, as

long as heating temperature does not exceed 500°C. When temperature in steel attains above 500°C, the strength after cooling starts to decrease gradually with the highest temperature steel ever reached (Neves et al. 1996). In addition, at this temperature level, stress-strain relation of steel rebar also changes due to phase transition.

Relations for high-temperature mechanical properties of reinforcing steel, as given in Eurocode 2 and ASCE manual (Lie 1992) are presented in the Appendix A.

2.4.2.3 Deformation properties

In comparison to concrete, reinforcing steel experiences higher thermal expansion at elevated temperatures. The thermal expansion of steel at elevated temperatures can be evaluated using the coefficient of thermal expansion (CTE), which is defined as dimensional variation in unit length of reinforcing steel due to unit change in temperature. The variation of thermal strain as a function of temperature suggested by ASCE Manual (Lie 1992) is shown in Figure 2.12. Overall, CTE of reinforcing steel increases with the rise in temperature. However, in the range of 650-815°C, CTE decreases at elevated temperatures, and this is mainly attributed to molecular transformation in steel.

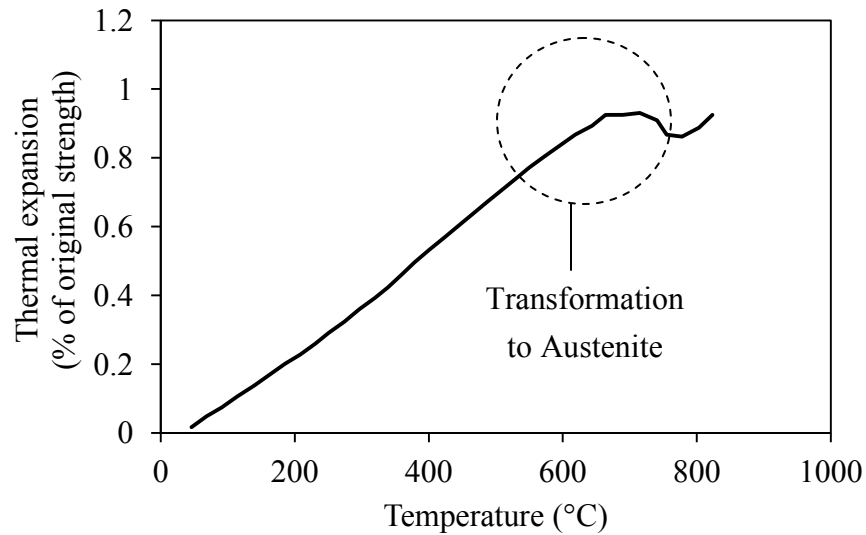


Figure 2.12 Variation of thermal expansion with temperature for reinforcing steel

Creep is another important variation to be considered for reinforcing steel at high temperatures. At room temperature, the creep of steel highly depends on its stress level, and creep strain increases at a very low pace. However, at high temperature (above 450°C), creep strain can be significant within a short time, due to the variation in crystal structures of steel. So far there is limited information found in the literature about the variation of creep strain with temperature for steel reinforcement. The available creep models, such as the one proposed by Harmathy (Harmathy 1967), are based on Dorn's theory, which relates creep strain to the temperature, stress, and time. More information on Harmathy's creep model is provided in Chapter 4.

2.4.3 FRP reinforcement

2.4.3.1 General

FRP materials are highly combustible and burn when exposed to fire. A large amount of combustible gases, ignite, release heat and propagate flame are generated

during burning of FRP. The emitted smoke, which affects visibility, hinders ability of the occupants to escape and poses difficulties for fire fighters to conduct evacuation operations and suppress the fire. Flammability, which is one of the indicators of fire hazard generally, refers to the tendency of a substance to ignite easily and burn rapidly with a flame. The flame spread and generation of toxic smoke, which are the two major concerns with FRP material, largely depend on the type of FRP formulation (composition). When used in buildings, structural members have to satisfy flame spread, smoke generation and fire resistance ratings prescribed in the building codes (Ahmed 2010).

For evaluating flame spread and smoke generation, ASTM recommends three different standard tests. ASTM E84 (2013) specify procedures for relative burning behavior of a building material by measuring flame spread index (FSI) and smoke density index (SDI). ASTM E662 (2013) specifies optical density test to measure characteristics of smoke concentration, while ASTM E162 (2013) describes test procedures for measuring and comparing surface flammability of different building materials when exposed to radiant heat energy. Generally, FRP manufacturers list their products for smoke generation and flame spread classifications in directories after getting specified tests from the specialized. Thus, in this research, it is assumed that FRP's have met the relevant flame spread and smoke generation rating specified in building codes and standards.

From the point view of structural fire engineering, the variations of thermal, mechanical, and deformation properties of FRP are more concerned, since they significantly influence the load resistance capacity of FRP strengthened RC members.

Currently, a wide range of FRP products are available in the market and any small changes in the composition of FRP (matrix or fiber) can influence their high temperature properties. Thus it is difficult to quantify the variation of each FRP product at elevated temperatures. This section reviews thermal, mechanical, and deformation properties of some primary FRP products in civil engineering applications, and these properties are critical to simulate thermal and structural behavior of concrete members incorporated with FRP under fire conditions.

2.4.3.2 Thermal properties

The influence of thermal properties of FRP to fire response of structural members depends on the type and amount of FRP reinforcement in use. For concrete members wrapped with external FRP laminates, FRP laminates might cover much of the surface of concrete members. When exposed to fire, FRP laminates essentially transforms to a char layer. Thus the charring from FRP laminates can provide certain level of thermal protection for original concrete members. In this case, thermal properties of FRP can significantly affect heat propagation within the concrete member. However, when FRP is used as internal reinforcement or as NSM strengthening, the influence of thermal properties of FRP is usually negligible, due to its small cross-sectional area as compared to concrete section. In these two case, FRP reinforcement can be handled in the same way as reinforcing steel in concrete members.

Thermal properties of FRP, which mainly refer to thermal conductivity, specific heat and density, vary significantly at elevated temperatures. There is very limited information on the variation of thermal properties of FRP at elevated temperatures,

especially for the temperatures above 400°C. Griffis et al. (1984) expressed the temperature dependence of thermal conductivity of carbon/epoxy composites as starting at an initial value of approximately 1.4 W/m-K, decreasing to about 0.2 W/m-K by 500°C, as shown in Figure 2.13. After this point, thermal conductivity of FRP remained almost constant with increased temperatures.

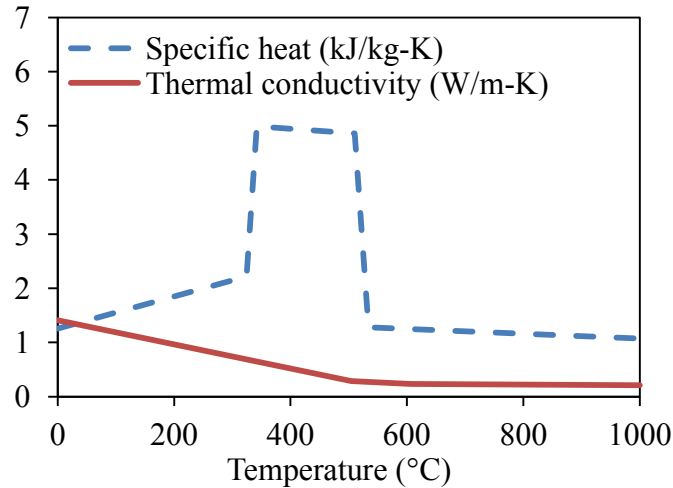


Figure 2.13 Variation of thermal properties with temperature for FRP

Specific heat is another critical parameter that influences heat transfer. Kalagiannakis and Van Hemdriek (2003) reported specific heat of 0.8 kJ/kg-K for both glass and carbon/epoxy FRPs at room temperature. Specific heat for both types of FRP increased with temperature, and reached 1.45 kJ/kg-K for carbon and 1.3 kJ/kg-K for glass at 170°C. Evseeva et al. (2003) reported a specific heat of 1.0 kJ/kg-K at 0°C, increasing to 1.5 kJ/kg-K at 100°C for phosphorous carbon/epoxy FRP material. Griffis et al. (1984) reported specific heat data for carbon/epoxy FRP used in aerospace applications which varied over a much wider range. The suggested property-temperature relation on thermal conductivity and specific heat of FRP, as a function of temperature, is compiled in Figure 2.13.

2.4.3.3 Mechanical properties

FRP is highly susceptible to temperature effects. It is known that most fibers are capable of maintaining strength at relatively high temperature, while it is polymer matrix component in FRP composites are vulnerable even at moderate temperatures. Thus, when polymer matrix experiences phase change (glass to rubber state, or rubber to leathery state), the mechanical properties of FRP degrade significantly.

There have been a few studies on mechanical properties of FRP at elevated temperatures. Kumahara et al. (1993) studied tensile strength and elastic modulus of FRP rebars at elevated temperature and residual strength after cooling. The test results indicated that at 400°C, the strength of aramid rebars dropped to 20% of their original values, and glass fiber bars with a vinyl ester binder retained relatively higher portion of original strength (40%). While carbon/epoxy bars did not lose strength until 250°C. For the residual strength after cooling, aramid bars was able to recover most of the original strength if AFRP temperature was within 150°C, while glass and carbon bars regained most of their strength even when they were heated to 250°C.

Fujisaki et al. (1993) tested carbon/vinyl ester FRP grids in tension under both stationary and non-stationary thermal regimes. Tensile strength of CFRP declined at around 100°C. When reaching 250°C, CFRP maintained 60% of its original strength. In the residual strength tests, negligible loss was observed for temperature up to 250°C. This study shows that FRP might retain most of its strength at moderately high temperatures, and FRP may possess high resilient strength.

Bisby et al. (2005) compiled temperature dependant strength and stiffness of FRP from a number of studies and proposed empirical equations to describe the

strength/stiffness degradation. These equations were assumed to fit a sigmoid function, and they reflected the variation of strength and stiffness with temperature. The proposed relations for strength and modulus of FRP ($f_{f,T}$ and $E_{f,T}$) at a given temperature T were given as follows.

$$f_{f,T} = f_{20^\circ C} \left(\frac{1-a_\sigma}{2} \right) \tanh(-b_\sigma(T-c_\sigma) + \frac{1+a_\sigma}{2}) \quad (2.3)$$

$$E_{f,T} = E_{20^\circ C} \left(\frac{1-a_E}{2} \right) \tanh(-b_E(T-c_E) + \frac{1+a_E}{2}) \quad (2.4)$$

where, $f_{20^\circ C}$ and $E_{20^\circ C}$ are the original stress and elastic modulus of FRP at room temperature respectively. a_σ , b_σ , c_σ , a_E , b_E , and c_E are the coefficients obtained from curve-fitting.

Wang et al. (2007) carried out an experimental study on high temperature strength degradation on FRP bars used as internal reinforcement. Totally 57 tension tests at various temperatures were conducted. The test results indicated that carbon and glass FRP lose 50% of their original strength at 325°C and 250°C, respectively. Modulus of elasticity of FRP showed negligible loss up to about 400°C, and then started to decrease rapidly beyond 400°C.

The above review indicates that previous studies on mechanical properties of FRP mainly focused on those of FRP laminates or internal rebars. There are no specific studies on high temperature properties of NSM FRP. Due to wide variety in shape (strip and rod) and composition (fiber volume, epoxy type), FRP reinforcement used for NSM strengthening might be significantly different from the above properties. Therefore, additional information on high temperature strength and stiffness properties of NSM FRP

is required to obtain reliable assessment on fire performance of NSM FRP strengthened beams.

2.4.3.4 Deformation properties

Under fire conditions, temperature induced thermal expansion in FRP reinforcement can also influence the behavior of concrete members incorporated with FRP reinforcement. Typically the coefficient of thermal expansion (CTE) of FRP varies in longitudinal and transverse directions. The longitudinal coefficient of thermal expansion is dominated by the properties of fibers, while the transverse coefficient is dominated by the properties of resin (Bank 1993). The values of CTE are also significantly different for various types of fiber, resin, and volume fraction of fiber. At ambient conditions, ACI 440.1 Guide (2006) provides some CTE values in longitudinal and transverse directions for different types of FRP rebars. However, at elevated temperatures, there is limited information on the variation of CTE. Some notable studies on high temperature thermal expansion of FRP are summarized in Table 2.1.

Since thermal expansion in longitudinal direction is a main factor that affects the effective stress in FRP, the discussion herein focuses on thermal strain in longitudinal direction. From Table 2.1, it can be seen in 20-200°C temperature range, the CTE of CFRP is quite small and fluctuates around zero, while CTE of GFRP reaches around 15×10^{-6} /K. This is because glass fibers experience much higher expansion than that in carbon fibers. In 200-800°C range, there is lack of test data on thermal expansion of FRP, this is attributed the fact that polymer matrix starts melting beyond 200°C and it is difficult to measure CTE of FRP as a whole piece. Based on the results of theoretical

studies (Schaery 1968, Nomura and Ball 1993), CTE values for CFRP and GFRP reinforcement, in the temperature range of 20-1000°C, can be assumed to be $5 \times 10^{-6}/K$ and $15 \times 10^{-6}/K$ respectively to account for temperature induced thermal strain.

Except thermal expansion, creep can also have critical influence on structural behavior of FRP when exposed to fire, since high temperature significantly accelerates creep strain and leads to relatively large deformation in FRP (Williams 2007). Generally, creep behavior of FRP is mainly dependent on the behavior of matrix materials. A cross-linked thermoset matrix exhibits less creep than thermoplastics. Fiber orientation might be another factor influencing the magnitude of creep in FRP. When fibers are in the loading direction, creep in fibers highly affects deformation of the entire composite.

Since FRP experience softening and melting at high temperatures, it is extremely difficult to evaluate creep strain of FRP at high temperatures. Rahman et al. (1993) conducted tensile creep tests on uniaxial carbon/glass hybrid FRP with 40% ultimate stress level at room temperatures. Data from the tests indicates that creep in the fiber direction is only 1.8% of the initial strain. However, at elevated temperatures, the creep strain can get enhanced. Raghavan and Meshii (1997) conducted experimental studies on creep behavior of carbon fiber-reinforced polymer at various stress levels and in temperature range of 20-150°C. The study shows that at the same stress levels, CFRP composite experienced twice the creep effect at 150°C as that at room temperature. The combination of high stress and high temperature makes creep strain very significantly. Based on the experimental study results, Raghavan and Meshii (1997) proposed the following to predict creep of FRP composites at elevated temperatures.

$$\varepsilon_{crf} = \int_0^t B \sigma^{0.01} e^{-H/kT} \sinh\left(\frac{\sigma}{kT}\right) dt \quad (2.5)$$

where, $B = 2.03 \times 10^{-4} T^{1.55} \cdot t^{0.25}$ (2.6)

ε_{crf} is creep strain of FRP, T is FRP temperature (K), σ is the stress in FRP (MPa), t is the fire exposure time (s), and k is Boltzmann's constant. H is the activation energy whose value follows the reported experimental data.

2.4.3.5 Bond properties

Bond plays a vital role in transfer of loads (forces) from concrete to FRP reinforcement. Depending on type of FRP reinforcement in use (internal rebar, external laminates, NSM strip), bond mechanism between FRP and concrete can be significantly different. In concrete members strengthened with external or NSM FRP reinforcement, the bond is generated through another intermediate adhesive layer applied between FRP and concrete, and the bond strength is essentially the ultimate shear strength developed in the adhesive materials (epoxy or cement mortar). While for concrete members reinforced with internal FRP bars, the bond mainly rely on the interlock action between deformed rebar and concrete. In light of these differences on bond mechanism, this section provides a review of high temperature bond properties between FRP and concrete for each individual case.

Table 2.1 Thermal expansion of FRP reinforcement reported in previous studies

Reference	Type of study	Material	Temp. (°C)	Longitudinal (10^{-6} / °C)	Transverse (10^{-6} / °C)
Gentry and Hudak (1996)	Experimental	Smooth glass/vinylester composite rod	20	4.8	38
		Glass/vinylester rebar with helical overwrap	20	8.2	32
		Composite rebar with molded reinforcing lugs	20	7.5	44
Nomura and Ball (1993)	Analytical	T300/5208 (graphite-epoxy)	25	-0.019	22.48
		T300/934 (graphite-epoxy)	25	0.239	27.86
		T100/2024 (graphite-epoxy)	25	1.64	27.59
		Metal matrix composites	25-815	3.5-4.6	--
Bowles and Tompkins (1988)	Analytical	T300/5208 (graphite-epoxy)	25	-0.113	25.23
		T300/934 (graphite-epoxy)	25	-0.002	29.03
		T100/2024 (graphite-epoxy)	25	1.44	26.12
	Experimental	T300/5208 (graphite-epoxy)	-183-149	-2-16	--
Anagnostopoulos (2008)	Analytical	LTM217 epoxy /Kevlar aramid fiber composite	25-100	0.2-0.7	--
Gorji and Mirzadeh (1989)	Analytical	Boron-epoxy composite	20	5.29	30.38
Foye (1975)	Experimental	Carbon-epoxy	20	5.42	30.7
Ishikawa (1979)	Experimental	Carbon-epoxy	20-150	-4-0	--
Pirgon (1973)	Experimental	CFRP	0-130	-10-5	--
ACI 440.1 (2006)	Experimental	AFRP rebar	20	-6 to -2	60-80
		GFRP rebar	20	6-10	21-23
		CFRP rebar	20	-9-0	74-104

Katz et al. (1999) studied bond properties of concrete members reinforced with internal FRP rebars using a number of commercially available FRP rebars, in temperature range of 20-250°C. Test results show a reduction of 80-90% in bond strength when the temperature increased from 20 to 250°C, while the conventional deformed steel rebars only showed a reduction of 38% of original bond strength in the same temperature range. A reduction in bond stiffness, which was determined from the slope of the ascending branch of pullout load-slip curve, was also observed with increase in temperature. The authors pointed out that bond properties between FRP rebar and concrete are highly sensitive to high temperatures, and the degradation of bond strength at elevated temperature relies mainly on polymer treatment at the surface of FRP rebar. Based on these experimental results, Katz and Berman (2000) proposed the following empirical relation to predict the degradation of bond properties at elevated temperatures.

$$\tau = 0.5(1 - \tau_r) \tanh \left\{ -\frac{0.02}{Cr} \left[T - k_1 \left(T_g + \frac{k_1}{0.02} Cr \right) \right] \right\} + 0.5(1 + \tau_r) \quad (2.7)$$

$$k_1 = \begin{cases} 1, & T_g \leq 80, \\ 1 - 0.025(T_g - 80) & 80 < T_g < 120, \\ 0 & T_g \geq 120 \end{cases} \quad (2.8)$$

where, τ is the normalized bond strength, T is the temperature, τ_r is the residual bond strength, Cr is the degree of cross-linking, T_g is the glass transition temperature of polymer.

As to RC members strengthened with external FRP laminates, there are a few studies on thermal effect to bond properties between FRP laminates and concrete (Blontrock et al. 2002, Di Tommaso et al. 2001, Klammer et al. 2005b, Leone et al. 2009,

Wu et al. 2004). Ahmed (2010) compiled the available test data on bond degradation in externally bonded FRP, and they are plotted in Figure 2.14. These data was mainly obtained from previous double-lap shear tests conducted on CFRP laminates bonded to concrete with adhesive. It can be noticed that these test data is pretty scattered, and this is because of the variation of FRP and adhesive materials used in different tests. Results from these tests indicate that bond strength degradation is negligible at low temperatures (around 40°C). However, significant reduction in bond strength was observed at temperatures beyond T_g . An empirical relation on variation of bond strength with temperature was also proposed as follows.

$$f_T = f_{20} (T \leq 40^\circ\text{C}) \quad (2.9)$$

$$\frac{f_T}{f_{20}} = 1 - \left(\frac{1}{80} \right) (T - 40) \quad (40^\circ\text{C} \leq T \leq 120^\circ\text{C}) \quad (2.10)$$

where, f_{20} and f_T are the bond strength at room and higher temperatures respectively, T is the temperature at the interface of FRP and concrete.

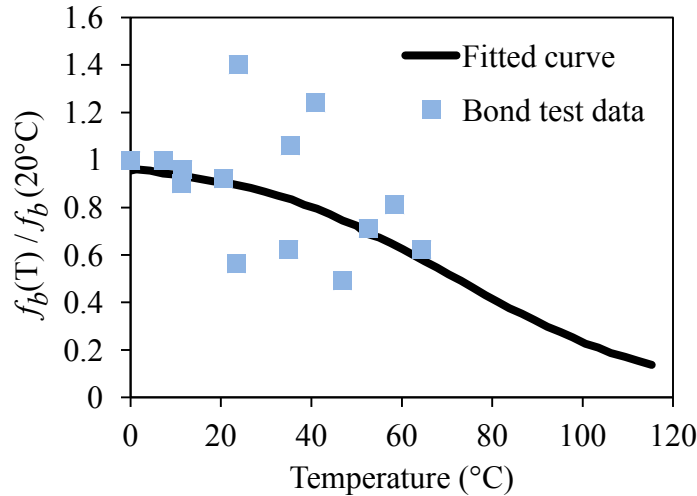


Figure 2.14 Variation of bond strength with temperature for externally bonded FRP

For concrete members strengthened with NSM FRP, Palmieri et al. (2011) conducted high temperature bond tests on NSM FRP systems. A series of 18 pull-out bond tests were performed on NSM FRP strengthened concrete blocks. Three types of FRP reinforcement were used for NSM strengthening (CFRP rod, CFRP strip, and GFRP rod), and the temperatures in the test was in a range of 20-100°C. Based on the test results, bond strength of NSM strengthening system barely decrease until the temperature in adhesive exceeds its glass transition temperature. When temperature increased beyond T_g , the failure mode of NSM bond changed from splitting of resin (50°C) to pulling out of FRP reinforcement (100°C). Also, strains along the bonded length became more uniformly distributed and the transfer length increases.

In summary, it can be seen there is limited research on bond properties between FRP and concrete, especially for NSM FRP system. Due to critical influence of bond properties on the behavior of the strengthened member, extensive research is still needed to identify the main factors that influence the bond behavior, and to develop a reliable relation to predict the deterioration of bond properties at high temperatures.

2.4.4 Fire insulation

Fire insulation is often applied to steel and wood structural members in buildings to enhance fire resistance. Concrete structures are not usually required to be protected with insulation due to its excellent inherent temperature resistance properties. However, based on results from previous study (Blontrock et al. 2000, Kodur et al. 2006, Williams et al. 2007, Ahmed and Kodur 2010), fire insulation is necessary for FRP strengthened RC members to maintain the strength and integrity of FRP reinforcement, due to

susceptibility of FRP to fire exposure. This section provides a brief review of commercially available insulation materials used for fire protection.

Literature studies indicate there are two main categories of fire insulation materials, insulation board (or mats) and sprayed insulation. Insulation board usually consists of calcium silicate, gypsum and vermiculite. This type of insulation is typically used to protect structural steel and aluminum, and it can provide thermal protection through its low thermal conductivity (0.12-0.16 W/m-K) and also through the water vapor which was trapped within the board during heating (BNZ Materials, 1998). For example, gypsum board is a fire insulation product that has been widely used in building applications. One reason is that gypsum board has low thermal conductivity of 0.16 W/m-k, which can significantly reduce conductive heat transfer in the insulation layer. Moreover, moisture content within gypsum board absorbs large amount of heat during evaporation process, and this also reduces heat energy passing through the board.

Spray-applied fire proofing is another commonly used insulation type. This type of proof usually comprise of some low thermal conductivity material (e.g. vermiculite) and a Portland cement or gypsum binder (Williams 2004). These materials are mixed with water and then sprayed to the surface of structural members. Depending on its specific composition, the sprayed fire proofing can achieve a low thermal conductivity of 0.043-0.078 W/m-K (Isolatek 2004). However, due to their light weight characteristics, thermal capacity of sprayed materials is usually small.

Due to relatively large porosity (especially for sprayed insulation), thermal properties of fire insulation often vary significantly with temperatures rise. For simulation purpose, the variation of thermal capacity and thermal conductivity at elevated

temperatures needs to be known. However, there is limited information on high temperature properties of insulation and the procedures for undertaking high temperature property tests. Bisby (2003) performed thermogravimetric analysis on VG insulation and proposed temperature-property relations in the range of 20-800°C, as shown in Figure 2.15. It can be seen that thermal conductivity of VG insulation initially decreases with increase in temperature (up to 200°C), and then remains almost constant till 500°C. Thereafter the thermal conductivity slightly increases with temperature. While the thermal capacity of VG insulation of insulation mostly remains in a stable level in the entire temperature range. The only exception is the peak at about 100°C, and this is due to evaporation of trapped water which consumes most of heat energy.

There are a great number of fire insulation materials available in the market. The thermal properties of some commonly used fire insulation materials are summarized in Table 2.2. However, there is lack of data on the variation of thermal properties of fire insulation with temperature.

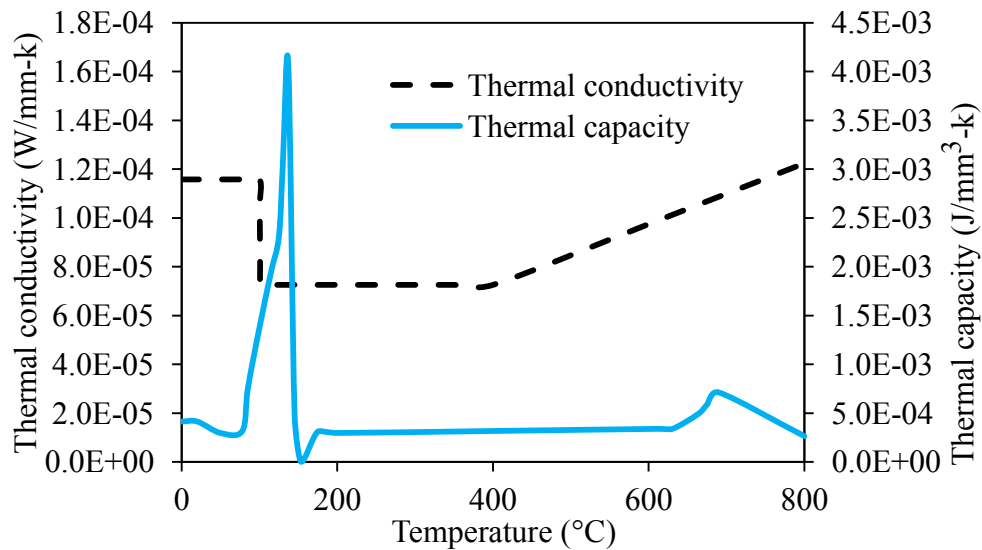


Figure 2.15 Variation of thermal properties with temperature for VG insulation

Table 2.2 Comparison of thermal properties for different fire insulation

Insulation type	Thermal conductivity (W/m-K)	Specific heat (kJ/kg-K)	CTE ($10^{-6}/K$)	Density (kg/m^3)
Tyfo Vermiculite-Gypsum (VG) (Bisby, 2003)	0.1158	1.1763	--	351
Vermiculite (www.vermiculite.org)	0.064	0.84 to 1.08	--	80-96
CAFCO 300 insulation (www.cafco.com)	0.078	--	--	240
Gypsum board (Manzello 2008)	0.25-0.32	0.9-1.0	7.22	711-805

2.5 Fire response of Concrete Beams Incorporated with FRP Reinforcement

FRP reinforcement can be used in concrete structural members in a number of ways. FRP rebar can be used as primary internal reinforcement in concrete beams or slabs; FRP laminates are usually applied for external strengthening in concrete beams or as confining for concrete columns; FRP strips and rods can be used as NSM strengthening in flexural concrete members. In this section, a brief review on fire performance of concrete beams incorporated with FRP reinforcement is provided. Based on the types and function of FRP reinforcement, this section separately discusses the fire response of concrete beams reinforced with internal FRP rebars, concrete beams strengthened with external FRP laminates, and those strengthened with NSM FRP reinforcement.

2.5.1 Concrete beams reinforced with internal FRP rebars

There are limited studies in the literature on fire performance of concrete beams reinforced with FRP rebars, and current design standards do not provide guidelines on

fire resistance of this type of beams (ACI 440.1 2006). Some of notable studies relating to fire resistance of concrete beams reinforced with FRP bars are reviewed here (See Tables 2.3 and 2.4).

Sakashita et al. (1997) carried out fire tests on 11 concrete beams reinforced with different types of FRP rebars. The tested beams were categorized based on fiber type (aramid, glass or carbon) and fabrication method (spiral, straight or braided) of FRP rebar. The test results indicated that beams reinforced with CFRP rebars achieved the highest fire resistance, followed by the beams reinforced with GFRP rebars, and then the beams reinforced with AFRP rebars. Also, the beams with spiral or straight fiber rebars yielded longer fire resistance than those with braided fiber rebars. Through these comparisons, the authors concluded that concrete beams with FRP rebars might achieve similar fire resistance as that of conventional steel reinforced concrete beams. However, room-temperature strength capacity of these beams were not clearly stated in the published paper (strength of steel and FRP rebars is not specified), and thus the fire resistance might not have been evaluated under the same loading level (load/capacity ratio).

Abbasi and Hogg (2006) carried out fire tests on two full-scaled concrete beams (350×400 mm) reinforced with different types of GFRP rebars. The parameters considered in the tests included resin type (thermoset and thermoplastic) and rebar size (one beam with only #4 bars, the other with #3, #4 and #6 bars) and shear stirrup (GFRP and steel stirrup). In the fire tests, both beams exhibited a long plateau on their load-deflection response, and both beams failed abruptly due to debonding of FRP rebars with surrounding concrete. The beam with thermoset FRP achieved a fire resistance of 128 minutes, while the beam with thermoplastic FRP achieved a fire resistance of 94 minutes.

Thus the authors concluded that with sufficient concrete cover, concrete beams with GFRP rebars can provide the required fire resistance ratings. Unfortunately, not many detailed test measurements (e.g. temperatures in rebars) are presented in the paper, and thus the test data is of limited use for validation of numerical models. Also, the applied load level on the beam during fire tests (ratio of applied load to room temperature capacity) was smaller than that encountered in practical situations.

Rafi et al. (2007) carried out fire tests on two simply supported beams reinforced with CFRP rebars (120×200 mm) under ISO 834 standard fire conditions. Both beams were tested under a load corresponding to 40% of room-temperature capacity. In the fire test, the temperature in CFRP rebar exceeded 500°C at around 50 minutes, and the resin of CFRP rebar got evaporated (indicated by the remaining cracked beams). The two beams achieved fire resistance of 51 and 63 minutes respectively, and prior to failure, carbon fibers (in rebars) supported the beam through a “tie-arch” mechanism. Therefore, the authors concluded that a concrete beam reinforced with CFRP rebars can perform equally well under fire conditions, as compared to steel reinforced concrete beam, and the anchorage at the two ends of rebars is vital to develop tie-arch mechanism under fire conditions.

Besides fire tests, limited numerical studies have been carried out on the fire performance of concrete beams reinforced with FRP rebars. Two main approaches were applied in these numerical studies. In the first approach, ACI 440.1 specifications (2006) are applied to check flexural and shear capacity of the critical section of the beam under fire conditions (Saafi 2002, Abbasi and Hogg 2005). This sectional analysis is similar to that of room-temperature capacity evaluation, but strength reduction factors for concrete,

steel and FRP reinforcement (resulting from high temperature) are applied in evaluating moment (or shear) capacity at a given fire exposure time. Based on the results from these studies, Saafi indicated that a minimum concrete cover of 64 mm to FRP reinforcement is required to achieve a fire resistance of 2 hours (Saafi 2002). Also, Abbasi and Hogg (2005) concluded that beams reinforced with FRP rebars provide half the fire resistance of an equivalent concrete beam with steel reinforcement.

In the second approach, researchers (Rafi et al. 2008, Hawileh and Naser 2012) carried out finite element analysis for evaluating fire response of FRP or steel reinforced concrete beams utilizing commercial software packages such as ANSYS. Various response parameters, including cross-sectional temperature, position of neutral axis, and mid-span deflections were evaluated under fire conditions. However, thermal and creep effects of FRP reinforcement and temperature induced bond degradation at FRP-concrete interface, are not accounted for in the analysis. These factors can significantly influence the behavior of concrete beam with FRP rebars under fire conditions.

The above literature review indicates that there is limited information on fire response of concrete beams reinforced with FRP rebars. Most of the previous studies involved undertaking standard fire tests or simplified numerical approaches on concrete beams reinforced with FRP rebars to check the adequacy of beams to satisfy fire resistance ratings. The critical factors that influence fire resistance of RC beams, such as realistic fire scenario, load level, bond degradation, and restraint conditions, are not yet addressed.

Table 2.3 Experimental studies on fire response of concrete beams reinforced with internal FRP rebars

Reference	Dimension (mm)	FRP rebar		f_c' (MPa)	Loading	Results
		Type	Strength (MPa)			
Sakashita et al. (1997)	200×300 ×4800 (11 beams)	AFRP, GFRP CFRP, Steel	--	36	2-point load (24kN)	Fire resistance: CFRP RC beam>GFRP RC beam>AFRP RC beam, beam with spiral or straight fiber rebar> beam with braided rebar
Abbasi and Hogg (2006)	350×500 ×4250 (2 beams)	GFRP	586-760 (Beam 1) 1000 (Beam 2)	42	4-point load (10kN)	Both beams failed abruptly due to debonding of FRP rebars with concrete. Fire resistance: beam with thermoset FRP (128 mins), beam with thermoplastic FRP (94 mins)
Rafi et al. (2007)	120×200 ×1750 (3 beams)	CFRP Steel	1676 (CFRP) 530 (steel)	33-35	4-point load (24kN)	Fire resistance: 51 or 63 mins for CFRP RC beam, 79 mins for steel RC beam. Anchorage of rebar is vital to develop tied-arch mechanism under fire conditions.

Table 2.4 Numerical studies on fire response of concrete beams reinforced with internal FRP rebars

Reference	Numerical approach or model	Results
Saafi (2002)	Applied sectional analysis to check flexural and shear capacity of the beam under fire conditions	Minimum concrete cover for FRP rebar should be 64 mm
Abbasi and Hogg (2006)	Proposed a semi-empirical temperature profile and strength reduction factors.	Proposed an analytical method for predicting strength capacity of beam under fire conditions.
Rafi et al. (2008)	Carried out finite element analysis using ANSYS. Model was validated against their own test results.	Material properties used in model provided satisfactory simulation results. The location of neutral axis remained unchanged for FRP reinforced beam under fire conditions.
Hawileh and Naser (2012)	Carried out finite element analysis using ANSYS. Model was validated against test data by Abbasi and Hogg (2006)	The developed FE model can capture the behavior of RC beams under fire conditions. Concrete cover thickness and fire scenario have significant influence on fire response of FRP reinforced beam.

2.5.2 RC beams strengthened with external FRP laminates

Since the last decade, several researchers have studied the fire response of RC beams strengthened with external FRP. The fire resistance of external FRP strengthened RC beams was evaluated for various configurations and fire scenarios, and the critical factors influencing this fire resistance were also evaluated through experimental or numerical studies (Ahmed and Kodur 2011, Williams et al. 2008, Firmo et al. 2012). The review of these studies is presented as follows (See Tables 2.5 and 2.6).

Blontrock et al. (2000) tested two RC beams and six CFRP strengthened RC beams under ISO standard fire exposure to investigate the effect of temperature on bond degradation between FRP and concrete. The beams were provided with fire insulation of Promatech-H or Promatech-100. Fire test results showed that fire insulation is necessary to minimize strength loss in FRP and to maintain low deflections in the beam during fire exposure. Also, the authors concluded that it is critical to maintain the adhesive temperature below glass transition temperature (about 80-90°C) in order to keep an effective bond between FRP and concrete.

Williams et al. (2008) conducted fire tests on four full-scaled FRP strengthened T-beams. The beams were protected with different insulation systems, and were tested under service load while exposing to ASTM E119 standard fire. In these fire tests, T_g of FRP was reached in the early stages of fire (about 60-90 minutes), but this did not lead to failure of the beam based on strength or critical temperature (rebar temperature) limit state. The beams achieved four hours fire resistance rating under ASTM E119 fire exposure.

Ahmed and Kodur (2011) presented results from fire resistance experiments on five rectangular reinforced concrete beams. Four of these RC beams were tested after being strengthened with CFRP laminates and protected with fire insulation, while the remaining one was tested as a control RC beam. The beams were tested by exposing them to fire and service load (about 50% of room temperature capacity). The test variables included type of fire exposure, anchorage zone, insulation type, and restraint conditions. Fire test results indicated that anchorage configuration plays a critical role in limiting the deflections of the strengthened beam after debonding of the FRP occurs at $T_g \pm 10^\circ\text{C}$. The authors concluded that FRP-strengthened RC beams supplemented with insulation possess sufficient fire resistance under ASTM E119 standard fire or a design fire. It was also found that the fire-induced axial restraint force can significantly increase the fire resistance.

Firmino et al (2012) studied the efficiency of different fire protection systems through fire tests on CFRP strengthened RC beams. The fire protection systems comprised of calcium silicate boards and layers of vermiculite/perlite cement based mortar applied along the beam soffit. The anchorage zones of the CFRP laminates were particularly insulated to evaluate the benefits of this construction detail. Fire test results indicated that if the strengthening system were left unprotected, CFRP laminate debonded after 23 minutes into fire exposure. However, if the fire insulation was applied, the debonding time was significantly delayed (60-89 minutes for 25 mm fire insulation, 137-167 minutes for 40 mm fire insulation). The post-fire assessment indicated that CFRP laminates transform into a “cable” fixed at the anchorage zones. When one of the anchorage zones debonds, the entire strengthening system will fail totally.

Besides fire resistance test, some researchers also evaluated fire resistance of RC beams strengthened with external FRP through numerical studies. Williams et al. (2008) developed a 2-D heat transfer model that employs an explicit finite difference formulation and heat transfer equations to determine temperature at each time step. The model is capable of predicting temperature distribution in FRP-strengthened rectangular and T-shaped beams exposed to standard fire scenarios. The model is validated by comparing model predictions with full-scaled fire test conducted at National Research Council, Canada (Williams et al. 2008). The temperature predictions within beam cross section were reasonably good as compared to fire test data. However, the model underestimates the temperature at the interface of FRP and insulation for the entire fire duration. Further, this model does not account for strength degradation of beam with temperature, and thus fire resistance cannot be evaluated only using this thermal model.

Hawileh et al. (2009) used finite element software, ANSYS, to study the thermal and structural response of FRP-strengthened T-beam under standard fire exposure. The model was validated against measured data from fire test conducted by Williams et al. (2008), and the predictions have reasonable agreement with experimental data. However, the model does not account for several important factors such as various strain components due to thermal and creep effects, fire induced bond-slip at FRP-concrete interface, as well as the effect of fire induced axial restraint force in the analysis.

Ahmed and Kodur (2010) presented a numerical approach for modeling the bond degradation in fire exposed FRP-strengthened RC beams. The numerical procedure was incorporated into a macroscopic finite element model which is capable of accounting high temperature material properties, different fire scenarios, bond degradation and

failure limit states. The validity of the model was established by comparing predictions from the program with data from fire tests on FRP strengthened RC beams. Results from the analysis indicated that significant bond degradation occurs close to glass transition temperature of the adhesive. The time at which bond degradation occurs depends on the fire insulation thickness and glass transition temperature of the adhesive. However, variation of adhesive thickness does not significantly influence fire resistance of FRP-strengthened RC beams.

The above review indicates that external FRP strengthening system is highly susceptible to fire exposure. When the temperature in adhesive exceeds its T_g , the debonding mostly likely occurs between FRP laminate and concrete. Thus, the anchorage zones of FRP laminates are vital to maintain its strengthening effect. Also, it is evident that thermal insulation is necessary to achieve a satisfactory fire resistance for RC beams strengthened with external FRP laminates.

Table 2.5 Experimental studies on fire response of RC beams strengthened with external FRP laminates

Reference	Dimension (mm)	Steel rebar	FRP laminates	f_c' (MPa)	Load	Insulation	Fire	Results
Blontrock et al. (2000)	200×300 ×2850 (No.= 8)	2 ϕ 16mm (591MPa)	200×1.2mm CFRP (2800 MPa)	57	40.6 kN 2-point load	Plat or U-shape	ISO 834	Fire insulation is necessary during fire exposure. It is critical to maintain the adhesive temperature below glass transition temperature in order to keep an effective bond between FRP and concrete
Williams et al. (2008)	Flange: 1220 ×150 Web: 300×250 (L=3900)	2 ϕ 20mm (500MPa)	100×1 mm CFRP (745 MPa)	41	34 kN/m, UDL	U-shape	ASTM E119	T_g of FRP was reached in the early stages of fire, but the beam did not fail based on strength or critical temperature limit state. The beams achieved four hours fire resistance
Ahmed and Kodur (2011)	254×406 ×3660 (No.= 4)	3 ϕ 19mm (420MPa)	203×2 mm CFRP (834 MPa)	52	70 kN, 2-point load	U-shape	ASTM E119	Anchorage plays a critical role in limiting the deflections of the strengthened beam. FRP-strengthened beams with insulation possess sufficient fire resistance. Fire-induced axial restraint force can significantly increase the fire resistance.
Firmo et al. (2012)	100×120 ×1350 (No.=5)	4 ϕ 6mm (524MPa)	50×1.2 mm CFRP (2742 MPa)	32	10.2 or 16.3kN 3-point load	U-shape	ISO 834	CFRP laminate debonded after 23 minutes if without insulation. CFRP laminates transformed into a “cable” fixed at the anchorage zones. FRP strengthening failed when one anchorage zones debonded.

Table 2.6 Numerical studies on fire response of RC beams strengthened with external FRP laminates

Reference	Numerical approach or model	Results
Williams et al. (2008)	Use 2-D heat transfer model that employs finite difference method and heat transfer equations.	Temperature predictions are reasonably good as compared to fire test data, but the model underestimates the temperature at the interface of FRP and insulation.
Hawileh et al. (2010)	Use ANSYS to study the thermal and structural response of FRP-strengthened T-beam under standard fire exposure.	The predictions have reasonable agreement with the experimental data. However, the model does not account for several important factors such as bond, creep etc.
Ahmed and Kodur (2010)	Use a macroscopic finite element model which accounts for high temperature material properties, realistic fire scenarios, and bond degradation of FRP.	Significant bond degradation occurs close to glass transition temperature of the adhesive leading to initiation of FRP delamination.

Table 2.7 Experimental studies on fire response of RC beams strengthened with NSM FRP reinforcement

Reference	Cross section	Steel rebar	NSM FRP	f_c'	Load	Insulation	Fire	Results
Rein et al. (2007)	--	--	CFRP rod, 2500 MPa	--	--	Plat	real fire	NSM strengthening provided a better performance than EBR system. For the beam protected by intumescent coating, NSM FRP strengthening stayed in place. If protected by the gypsum board, NSM FRP strengthening remained intact.
Burke et al. (2012)	254×102 mm	2 ϕ 6, 667 MPa	CFRP strip, 2068 MPa	46 MPa	20 kN	no	200°C	NSM FRP with epoxy adhesive yielded much higher strength than those with cementitious grout at ambient temperature. But at high temperatures, the slabs with cementitious grout achieved higher duration than those with epoxy adhesive.
Palmieri et al. (2012)	200×300 mm	2 ϕ 16, 550 MPa	CFRP rod or strips	40 MPa	36 or 40.5 kN 2-point load	Plat or U-shape	ISO 834	Fire insulation fell off on some beams and NSM FRP reinforcement attained high temperatures. But all tested beams sustained service loads for at least 2 hours. A U-shaped fire protection is more efficient than that of a flat protection at the bottom surface of the beam only.

2.5.3 RC beams strengthened with NSM FRP reinforcement

Since NSM FRP strengthening is relatively a new technique for civil construction, the literature on fire response of NSM strengthened RC members is extremely scarce (See Table 2.7). Rein et al. (2007) performed fire tests to compare the fire performance of two different strengthening systems, EBR and NSM. For each type of strengthening, three specimens were fabricated. One was left unprotected, one was painted with an intumescent coating, and the remaining one was protected by a gypsum board box. The test results indicated that NSM FRP system had a better performance than EBR system. For the beam protected by intumescent coating, NSM FRP strengthening still stayed in place, although the adhesive was glazed and contained transverse cracks. While for one protected by the gypsum board, NSM FRP strengthening system remained intact in the grooves. However, this test did not record the temperature in the strengthened beams, and the loading was not specified in the literature either. Thus, this experimental study cannot provide comprehensive evaluation of fire response of NSM FRP strengthened RC beams.

Burke et al. (2013) tested 13 reinforced concrete slabs under elevated temperature conditions (up to 200°C, not fire exposure), 11 of which were strengthened in flexure with a single NSM FRP tape. Epoxy and cementitious grout were used on different slabs to study the influence of different adhesive on the behavior of NSM FRP system at both ambient and elevated temperatures. The test results indicated that provision of epoxy adhesive on NSM FRP reinforcement yielded much higher strength capacity as compared with cementitious grout at ambient temperature, and this was attributed to better bond from epoxy adhesive. However, at elevated temperatures (at about 200°C), the slabs with cementitious grout achieved higher duration (failure time) than those with epoxy

adhesive. Based on the test results, the authors inferred that insulated NSM FRP strengthened slabs provide required fire resistance for building applications.

Palmieri et al. (2012) conducted fire tests on ten RC beams strengthened with various NSM FRP configurations, in conjunction with fire insulation, to evaluate fire performance. In these fire tests, fire insulation on some of the beams fell off, and NSM FRP reinforcement attained very high temperatures (about 850°C). However, all tested beams sustained service loads for at least 2 hours under ISO 834 standard fire exposure. Also, it was found that a U-shaped fire protection (extending to the sides of the beam) is more efficient than that of a flat protection at the bottom surface of the beam only.

The above review clearly indicated that there are a number of knowledge gaps on fire response of NSM FRP strengthened RC beams. No experimental studies are conducted to evaluate fire performance of unprotected NSM FRP strengthened RC beams (without insulation). Further, no information is documented on the behavior of NSM strengthened RC T-beams under standard or realistic fire conditions. Also, no numerical studies are carried out to evaluate critical factors governing the fire response of NSM strengthened RC beams. Therefore, extensive studies are still required to develop a comprehensive understanding on the behavior of NSM FRP strengthened RC beams under fire conditions.

2.6 Codes and Standards for FRP strengthened RC members

Guidelines for design of FRP strengthened RC structures are available in various standards (ACI 440.2 2008, CSA S806 2002, Fib Bulletin 14 2007). In the latest version of ACI 440.2R guidelines (2008), design specifications on NSM FRP strengthening

system are incorporated, including size of NSM groove (refer to Figure 2.2), NSM bond strength, flexural strengthening design approach, etc. For evaluating flexural strength of NSM FRP strengthened RC members, ACI specification applies an approach analogy to that of external bonded FRP. A reduction factor for preventing debonding failure of NSM FRP is recommended on ultimate strain of FRP as follows.

$$\varepsilon_{fd} = 0.7 \varepsilon_{fu} \quad (2.11)$$

where ε_{fd} is debonding strain of FRP reinforcement, ε_{fu} is design rupture strain of FRP reinforcement. Utilizing Eq. 2.11, tensile strength NSM FRP can be obtained and the flexural strength capacity of RC member can be evaluated based on force equilibrium and strain compatibility principles. Also, a reduction strength factor of FRP ψ_f , which is in addition to the flexural strength reduction factor ϕ , is recommended to address reliability of FRP contribution to flexural strength.

Although providing design guidelines of NSM FRP strengthening at room temperature, current codes and standards do not specify fire design guidelines for FRP strengthened RC members, or simply neglect the strength contribution of FRP reinforcement in the event of fire. ACI 440.2R (2008) recommends that the nominal resistance of FRP strengthened RC member at elevated temperature $R_{n\theta}$ should satisfy the combination effect of dead load and live load, which is

$$R_{n\theta} \geq S_{DL} + D_{LL} \quad (2.12)$$

This resistance $R_{n\theta}$ does not account for the contribution of the FRP systems unless FRP temperature can be demonstrated to remain below a critical temperature for FRP. Also,

ACI 440.2 (2008) recommends that the lowest T_g of FRP or epoxy adhesive can be taken as the critical temperature of an FRP strengthening system. Similarly, FIB Bulletin 14 (2007) suggests that without fire protection, the contribution of FRP strengthening should be totally neglected. In the case of strengthened elements with fire protection, FRP strengthening is considered only when the adhesive temperature does not exceed the limit of 50-100°C. However, based on previous studies presented in Section 2.5, these design recommendations are overly conservative.

A review of current design guidelines in codes indicates that no specific fire design provisions exist for evaluating fire response of NSM FRP strengthened RC members due to lack of information. There are only limited guidelines for fire endurance of external FRP strengthened RC members, but they are too conservative. Due to superior performance of NSM FRP system under fire conditions, a different evaluation method for fire resistance should be updated in the codes and standards. Also, from the purpose of fire safety design, a rational design methodology is needed to simply and accurately access fire resistance of FRP strengthened concrete members.

2.7 Summary

Based on the above literature review, it is evident there is very limited information on the fire response of NSM FRP strengthened RC members. At material level, available test data on high temperature properties are mainly for external FRP laminates or internal FRP rebars, and they cannot be used for modeling fire performance of NSM FRP strengthened RC beams. At structure level, limited fire resistance tests have been carried out, and a number of key issues, such as fire resistance of unprotected RC

beams with NSM strengthening, fire resistance of NSM FRP strengthened T-beams, are not yet addressed. Further, there is no numerical model for predicting fire resistance of NSM FRP strengthened beam, and thus there is lack of effective tools used for parametric studies on some critical influencing factors. Due to these knowledge gaps, in current codes and standards, no specific provisions are provided for structural fire design of NSM FRP strengthened RC beams. Therefore, for widespread application of NSM FRP technique for strengthening of RC beams, comprehensive experimental and analytical studies are required for developing rational design methodologies on NSM FRP strengthened RC members.

CHAPTER 3

HIGH TEMPERATURE MATERIAL PROPERTY

3.1 General

For evaluating fire response of NSM FRP strengthened RC members, high temperature dependant properties of constituent materials, namely, concrete, reinforcing steel, NSM FRP, are required. The thermal and mechanical of concrete and reinforcing steel are well established. However, there is lack of data on properties of NSM FRP reinforcement at elevated temperatures. These properties, namely, tensile strength and elastic modulus, bond strength and modulus, and thermal expansion, are different from that of FRP used as internal and external reinforcement, due to the difference in composition and cross sectional shapes.

To generate data on high temperature material properties of NSM FRP at elevated temperatures, a series of tensile strength, bond strength, and thermal expansion tests were carried out. Data from these tests is utilized to develop empirical relations for tensile strength and modulus, bond strength and modulus, and thermal expansion of NSM FRP reinforcement over a wide temperature range.

3.2 Tensile Strength Tests

As part of experimental studies, a number of tension tests were carried out on NSM FRP strips and rods over a wide temperature range. Data from tests are utilized to

evaluate tensile strength and elastic modulus of NSM FRP at various temperatures. Details on test procedure and results are presented as follows.

3.2.1 Preparation of test specimens

The experimental program consisted of tension tests on 25 CFRP strips and CFRP rods at various temperatures. 13 of these test specimens were CFRP strips, while remaining 12 were CFRP rods. CFRP strips were of 4.5 mm thickness and 13.5 mm width, and CFRP rods were of 6.4 mm diameter. The nominal tensile strength and modulus of CFRP strip, as specified by the manufacturer, is 2790 MPa and 155 GPa respectively, and the ultimate strain is 0.018. For CFRP rod, the corresponding nominal tensile strength, elastic modulus and ultimate strain are 2070 MPa, 124 GPa and 0.017 respectively. CFRP specimens for tests were provided by FYFE Co. LLC. Other properties of FRP reinforcement used in the test program are given in Table 3.1.

Table 3.1 Properties of NSM CFRP reinforcement as specified by manufacturer

NSM reinforcement	Tensile properties			Dimension (mm)	Density (g/cm ³)	Fiber content (%)
	Strength (MPa)	Modulus (GPa)	Ultimate strain			
Strip	2790	155	0.018	13.5×4.5	1.81	62
Rod	2070	124	0.017	dia. 6.4	N/A	60

It is well established that CFRP reinforcement possesses high tensile strength at ambient conditions. However, in a tension test, two ends of CFRP are susceptible to crushing under the pressure of gripping. Thus strong anchors have to be provided at the two ends, to facilitate gripping of CFRP specimen. The provision of proper anchors ensures failure to occur in the central region of the specimen, rather than at ends (in the

anchorage zone). A specialized anchorage system was implemented while preparing CFRP strip/rod specimens for tension tests. The anchor system was developed following ACI 440.3 specifications (2006) and those recommended by Wang et al. (2007). This is achieved through filling high strength adhesive into a circular steel tube (confinement), as shown in Figure 3.1a.

In this experimental program, both high strength epoxy (Tyfo S epoxy) and expansive cement (RockFrac NEDA) were applied as filling materials to evaluate their relative bond performance. Tyfo S epoxy is a two-component matrix material used in bonding applications and is marketed by FYFE Co. LLC. This epoxy was prepared by adding component A (modified epoxy resin) to component B (hardener) in a volumetric ratio of 100:42 (or a weight ratio of 100:34.5). The added ingredients were mixed for 5 minutes using a mixer at a speed of 400-600 RPM until two components are uniformly blended. Another filling material used in the fabrication of anchorage system is RockFrac NEDA expansive cement, which is used as non-explosive demolition agent and is marketed by RockFrac Company. The cement mortar was prepared by adding RockFrac cement into cold water (30% of the overall weight), and then thoroughly mixing cement and water to get a uniform mortar.

Commercially available steel pipes were selected as confinement for filling materials, to ensure sufficient bond is generated between filling material and CFRP specimen. The nominal dimensions of steel pipes are 42 mm in outer diameter and 1.6 mm in thickness, and the pipes were cut into tubes of 356 mm length. These dimensions are as per recommendations of ACI 440.3 standard (2006) and previous researchers (Wang et al. 2007). To increase friction between filling material and tube, 102 mm long

thread was fabricated inside the surface of the tube. To prevent sliding between CFRP and filling material, some small dents were created on CFRP strip or rod, and steel wires were bound to these dents, as shown in Figure 3.1b. Through this procedure a higher interaction (friction) was generated between CFRP and anchor system.

When epoxy (or cement) is filled into the tube, CFRP strip or rod had to be aligned vertically and centrally in the steel tube, to avoid any eccentric forces generated during tension test. For this a steel frame was fabricated to align CFRP and tube in the vertical direction, as shown in Figure 3.1c. The steel tubes sit on a wooden board and they were clipped by two aluminum plates. A wooden plug, with a hole in the center, was installed at the bottom of the tube so that CFRP specimen can be placed centrally. CFRP specimen was also fixed at the top of steel frame to ensure it was aligned vertically. Once the epoxy gets hardened in the steel tube, CFRP specimen is turned around for casting anchor system at the other end.



(a) Epoxy filling



(b) Wires on FRPs



(c) Steel frame



(d) Test specimen

Figure 3.1 Fabrication of anchor system for FRP specimens

3.2.2 Test set-up

Room temperature tensile strength tests on NSM CFRP specimens were carried out using Hydraulic Materials Test System (MTS), since MTS machine is capable of providing high compression pressure to grip the two ends of test specimens, as well as applies higher tension load so as to reach high strength and stiffness of CFRP specimens at room temperature. CFRP strip and rod specimens for room temperature tests were specially prepared to fit MTS machine set-up. The test apparatus and specimens for room temperature tests are shown in Figure 3.2.

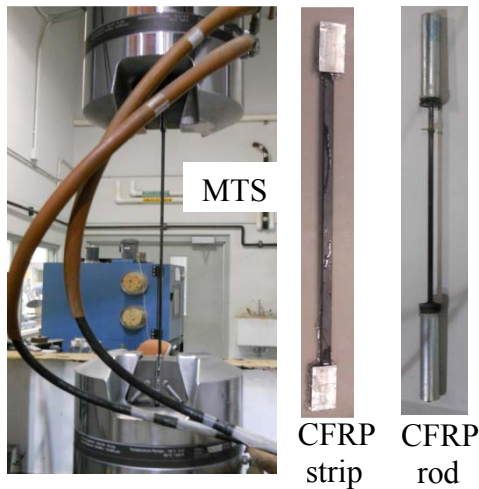


Figure 3.2 Test apparatus and specimens for room temperature test

For high temperature tests, a different test set-up was developed, and an illustration of this set-up is depicted in Figure 3.3. In this set-up, two ends of CFRP specimen (with anchor system), are clipped to two pairs of clamping brackets respectively, which are connected to top and bottom beams. The CFRP specimen is loaded in tension by adjusting the distance between these two beams. Two hydraulic jacks, sitting on the bottom steel beam, can directly apply specified loading to the top beam through an extension rod. When hydraulic jacks apply an increasing load, the top

beam moves upward and thus CFRP specimen gets stretched longitudinally. The top beam is always maintained perfectly horizontal to minimize eccentric loading occurring during the test. The heating device comprised of a small scale furnace which is placed between two pairs of clamping brackets. Through this set-up, tensile strength test can be conducted by heating the CFRP specimen to a desired temperature and then subjecting it to tensile loading.

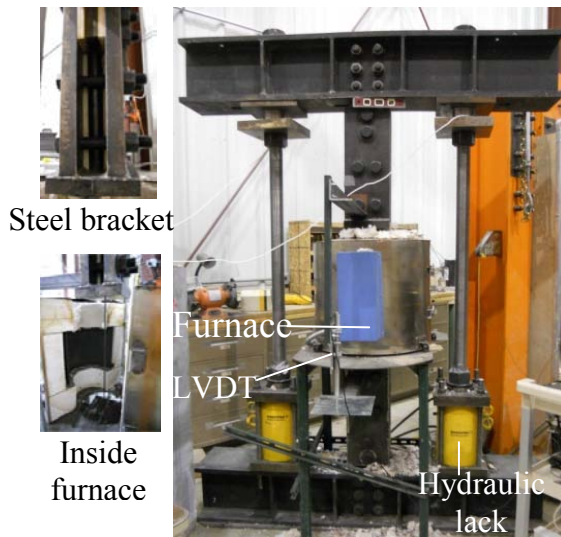


Figure 3.3 Test setup for FRP tension test at elevated temperatures

During the test, CFRP specimen is heated to a target temperature, and then the heating is continued for additional 20 to 30 minutes to ensure the entire specimen attains target temperature. To accurately monitor the temperature of CFRP specimen, two thermocouples are installed on the surface of CFRP specimen at two different locations (mid-height and quarter height), and the average of these two thermocouple readings is taken as the actual temperature of the specimen. The heating rate of furnace is set to be at 5-10°C/min, depending on the target temperature: a faster rate is used for higher target temperatures. The progression of measured CFRP temperature with time is shown in

Figure 3.4. It can be seen in the figure that in each case, temperature gradually increases to a target temperature, and then the specimen is maintained at this target temperature for about 20 minutes. This ensures that the specimen and furnace reach thermal equilibrium conditions and that the internal and surface temperatures of the specimen were sufficiently close to the target temperature.

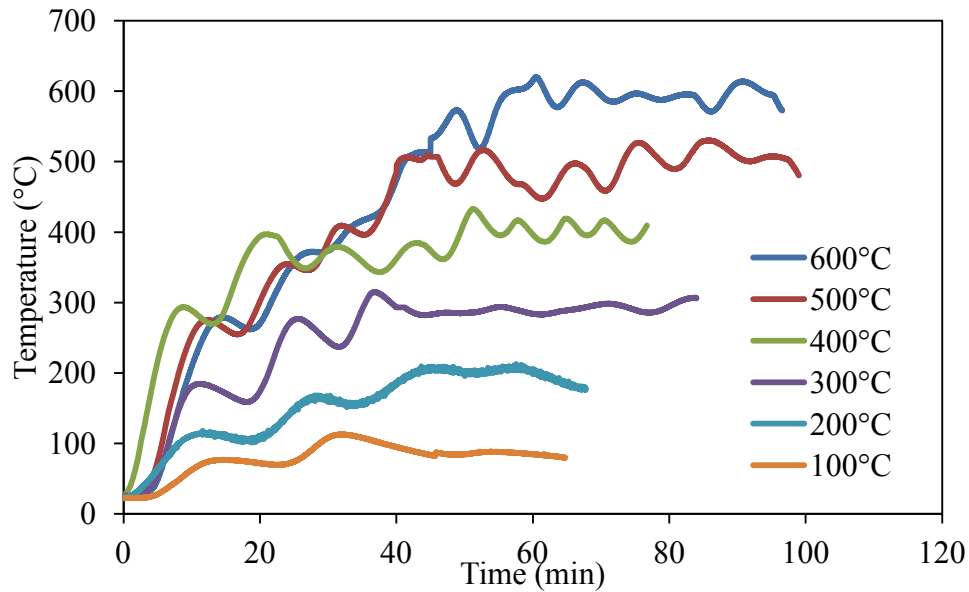


Figure 3.4 Temperature progression in FRP during high temperature tension tests

Following the specimen attaining a target temperature, tension test is carried out using hydraulic jacks. To measure elongation of CFRP in tension tests, a linear variable differential transformer (LVDT) is placed between the upper and the lower clamping brackets. The variation of distance between these two brackets is taken as elongation of CFRP specimen placed between two anchors, since the elongation of CFRP in anchor parts is negligible. The elongation measurements start as soon as loading is applied, and the displacement of the upper pair of brackets is recorded until CFRP specimen fails. The reliability of loading equipment and elongation measurements are verified through two

preliminary tests, one using steel strand and the other using CFRP strip. In these two tests, strain gauges were placed along the longitudinal direction of the specimen, and the measurement of strain gauges was compared with the readings from loading cell. As shown in Figure 3.5, in steel strand test, tensile stress in specimen kept increasing until steel entered yielding phase. While in CFRP strip test, tensile stress in specimen increased linearly. It can be seen that the stress values based on load reading match well with those obtained from strain gauges (product of strain and modulus), and thus the measurement from instrumentation is considered to be reliable.

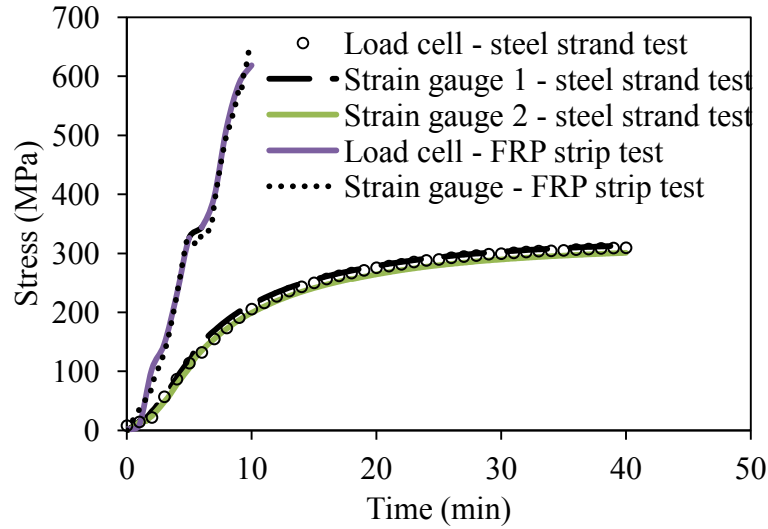


Figure 3.5 Comparison of measured stresses using loading cell with strain gauges

3.2.3 Results and discussion

Data recorded in tension tests is utilized to evaluate tensile strength and elastic modulus of NSM CFRP at various temperatures. The tensile strength was calculated by dividing the maximum load at failure by the actual cross-sectional area of test specimen, while elastic modulus was evaluated as the slope of linear part of stress-strain curve. At

each target temperature, two tension tests were conducted, and the average of two values was taken as tensile strength and elastic modulus of CFRP. Results from these tests at various temperatures are tabulated in Tables 3.2 and 3.3 for CFRP strips and rods respectively.

The tensile strength and elastic modulus of NSM CFRP strip, based on room temperature tests, were found to be 1641 MPa and 150.8 GPa respectively, and the corresponding values for CFRP rod are 1577 MPa and 130.9 GPa respectively. The measured room temperature elastic modulus of CFRP strips and rods are very close to those specified in manufacturer data (2.7% error for strip and 5.6% error for rod). However, room temperature tensile strength obtained from tension tests is relatively lower than manufacturer specified nominal strength. This is mainly attributed to the fact that CFRP resin fractures at a relatively low load. In the room temperature tests, failure of CFRP specimen gets initiated through cracking of resin. With increase in load, CFRP specimen gradually split into bunch of fibers, and some of these fibers were fractured or pulled out from anchors at the end. This resulted in drop in tension load due to reduction in the amount of fibers in a CFRP specimen. Although CFRP specimen does not break (fracture) totally, the peak tension load is attained when majority of resin cracks. In fact, the strength specified in the manufacturer data is essentially the strength of carbon fibers, but in tension test CFRP specimens hardly reach this strength due to fracture of resin. Thus, the tensile strength obtained in the test is taken as the actual room temperature strength of CFRP.

Results and observations from strength tests are tabulated in Tables 3.2 and 3.3. It can be seen in these two tables that strength and elastic modulus of NSM CFRP strip and

rod decrease with increase in temperatures. The variation of tensile strength of CFRP strip and rod with temperature is plotted in Figures 3.6a and 3.7a. The trends in both figures indicate that the degradation of tensile strength in CFRP can be grouped into three stages. In 20-200°C temperature range, tensile strength of CFRP decreases gradually at a slow pace, and CFRP strip and rod retain about 80% of original strength at 200°C. In current practice, CFRP is assumed to lose significant strength past its glass transition temperature (around 80°C). However, data from these strength tests clearly indicate that CFRP resin remains intact till about 200°C, and thus CFRP retains much of its initial strength. In 200-400°C temperature range, CFRP strip and rod experience faster degradation of their strength, and this is mainly due to decomposition of polymer resin at around 300°C. As noted from observations (see Tables 2 and 3), resin starts melting at 300°C, but does not get totally decomposed, hence CFRP splits into bunches of fibers and these fibers primarily resist tension load. Based on linear interpolation, the tensile strength of CFRP strips and rods drop to 50% of their original strength at about 305°C and 330°C respectively. This temperature can be treated as critical temperature for CFRP strip or rod. The critical temperature analogy used for conventional steel reinforcing bars is defined as the temperature at which steel loses 50% of its room temperature strength. In the third stage (400-600°C), majority of polymer resin gets decomposed, and only individual fibers contribute to load resistance. The strength of CFRP rod and strip degrades at a very high rate at this stage and reaches about 10% of their original strength. The amount of strength retention is highly dependent on the extent of oxidation of carbon fibers.

Table 3.2 Tensile strength and elastic modulus of CFRP strips at various temperatures

Temp. (°C)	Strength (MPa)	Average strength (MPa)	% of initial strength	Elastic modulus (GPa)	Average modulus (GPa)	% of initial modulus	Failure mode
20	1724	1641	100	141.2	150.8	100	CFRP split into a bunch of fibers, and then fibers got fractured or were pulled out
	1559			160.5			
100	1431	1461	89.0	160.1	135.9	90.1	CFRP split into a bunch of fibers, and then fibers got fractured around the mid-height
	1491			111.7			
200	1532	1122	76.2	137.5	123.5	81.9	CFRP split into a bunch of fibers, and fibers fractured around the mid-height
	968			109.5			
300	705	768	50.5	89.0	79.9	52.9	Resin melted and fibers fractured around the mid-height
	831			70.7			
400	585	717	43.7	51.5	70.0	46.4	Majority of resin got decomposed, fibers were stretched apart
	850			88.5			
500	657	512	31.2	8.1	33.1	22.0	Little resin left and fibers were stretched apart
	367			58.1			
600	175	175	10.6	--	--	--	No resin left, fibers got separated and stretched apart

Table 3.3 Tensile strength and elastic modulus of CFRP rods at various temperatures

Temp. (°C)	Strength (MPa)	Average strength (MPa)	% of initial strength	Elastic modulus (MPa)	Average modulus (MPa)	% of initial modulus	Failure mode
20	1536	1577	100	127.2	130.9	100	CFRP split into a bunch of fibers, and then fibers got fractured or were pulled out
	1618			134.6			
100	1536	1399	88.7	991.0	115.0	87.8	CFRP split into a bunch of fibers, and then fibers got fractured around the mid-height
	1261			130.9			
200	1199	1274	80.8	916.2	96.9	74.0	CFRP split into a bunch of fibers, and then fibers got fractured around the mid-height
	1349			102.1			
300	1114	927	58.8	105.3	93.9	71.7	Resin melted and fibers got fractured around the mid-height
	741			82.4			
400	484	484	30.7	69.4	69.4	53.0	Majority of resin got decomposed, fibers were stretched apart
500	223	312	19.8	65.9	52.9	40.4	Little resin left and fibers were stretched apart
	401			39.9			
600	126	126	8.0	26.6	26.6	2.0	No resin left, fibers got separated and stretched apart

It can be seen in Figures 3.6 and 3.7 that the measured strength and modulus data at elevated temperatures is relatively scattered as compared to those obtained at room temperature. This is mainly attributed to two factors, variation of heat flux in a specimen and sliding (slip) occurring between CFRP and anchors. Since heating rate of furnace is controlled manually, the heat flux introduced by furnace is different from one test to another, and this results in variation in specimen temperature at the time of test. Also, in some high temperature tests, there was slight sliding that occurred between CFRP and epoxy at the anchors, which also lead to variations in the measured strength. The use of

expansive cement in anchors generates higher bond performance as compared to that of epoxy, and only negligible slip occurred in specimens with expansive cement anchors.

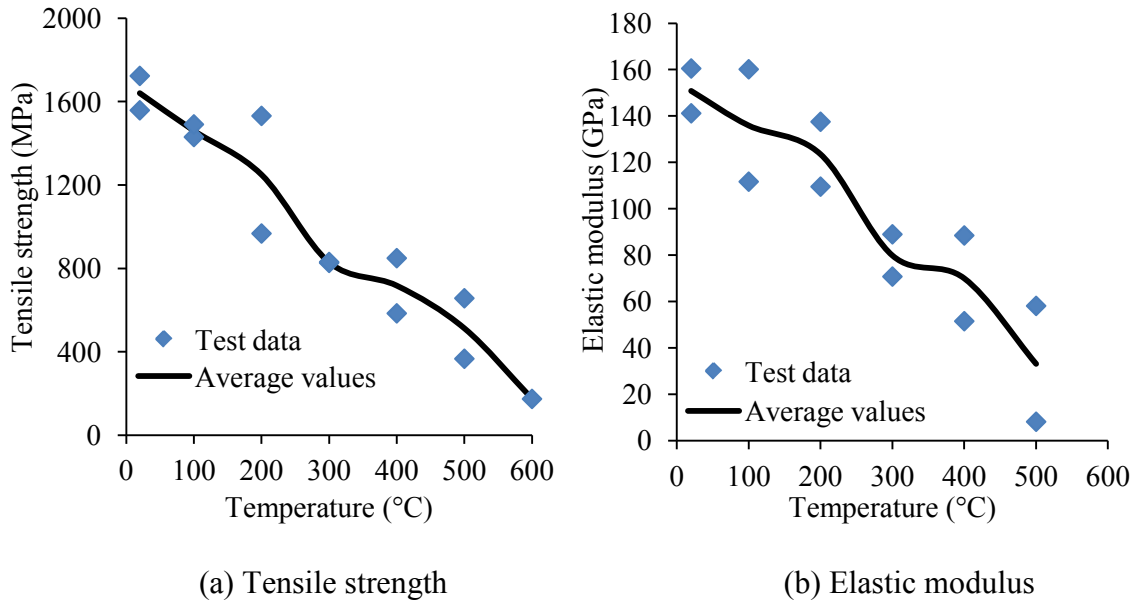


Figure 3.6 Variation of tensile strength and elastic modulus of CFRP strips with temperature

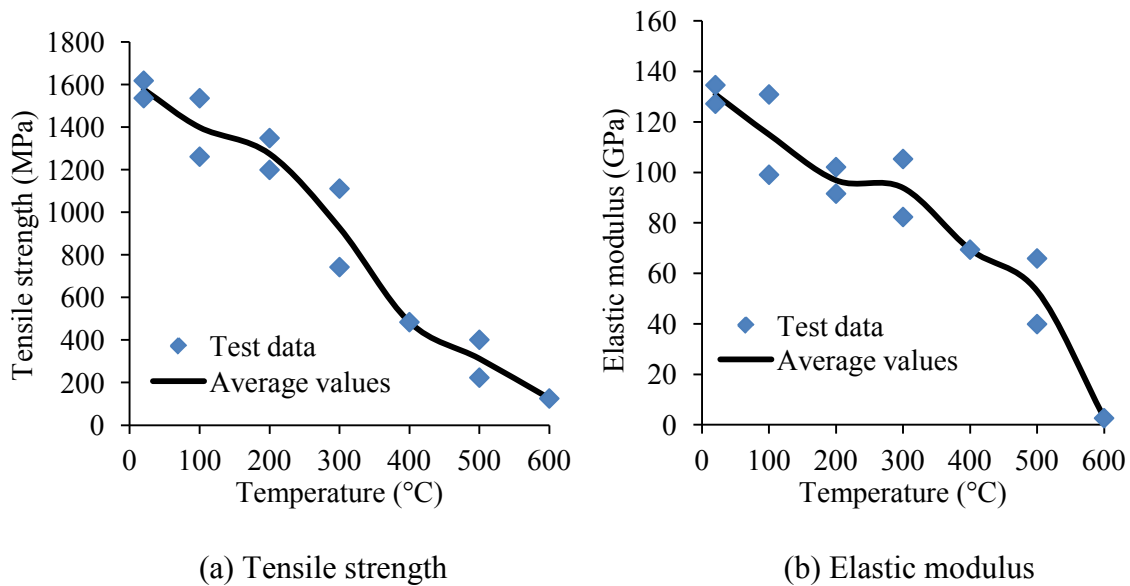


Figure 3.7 Variation of tensile strength and elastic modulus of CFRP rods with temperature

The stress-strain relationships for CFRP strips and rods at various temperatures are shown in Figures 3.8 and 3.9 respectively. It can be seen that CFRP strip and rod exhibit almost linear stress-strain response at both ambient and high temperatures. Also, the ultimate strain of CFRP decreases with increase in temperature. Thus the ductility of CFRP reinforcement decreases at higher temperatures, which is contrary to that occurring in conventional steel reinforcing bars. The slope of stress-strain curves at different temperatures is taken as the elastic modulus of CFRP specimens, and they are plotted in Figures 6b and 7b. It can be seen in Figures 6b and 7b that the decrease in elastic modulus follows similar trend as that of tensile strength. However, at most target temperatures, relatively higher percentage of elastic modulus is retained as compared to that of tensile strength. Based on the observations in tests, degradation of elastic modulus is more dependant on the state of polymer resin. Prior to decomposition of polymer resin (300°C), the integrity of CFRP specimen is well maintained, and thus higher level of elastic modulus is retained. Once polymer resin melts and evaporates, CFRP specimens turn into a bunch of separate fibers, and thus elastic modulus gets significantly reduced.

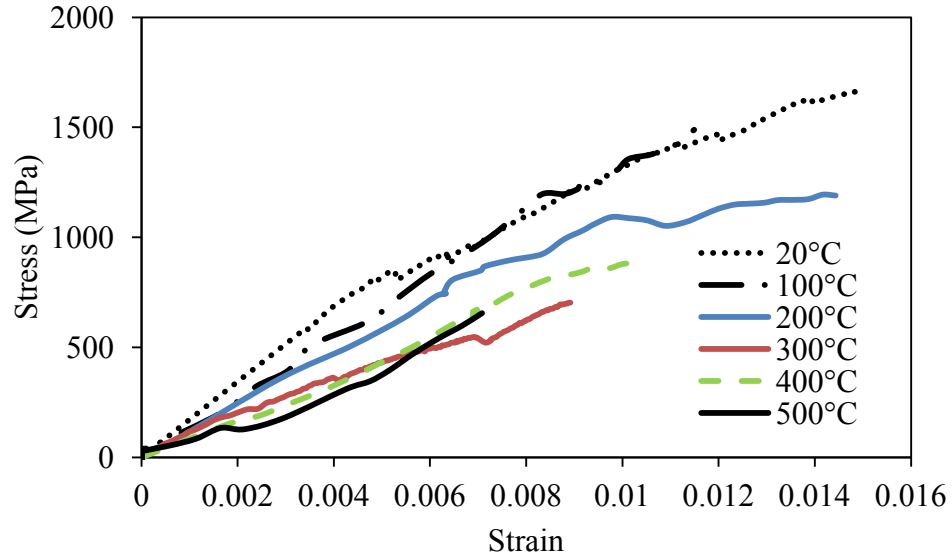


Figure 3.8 Stress-strain response of CFRP strips at various temperatures

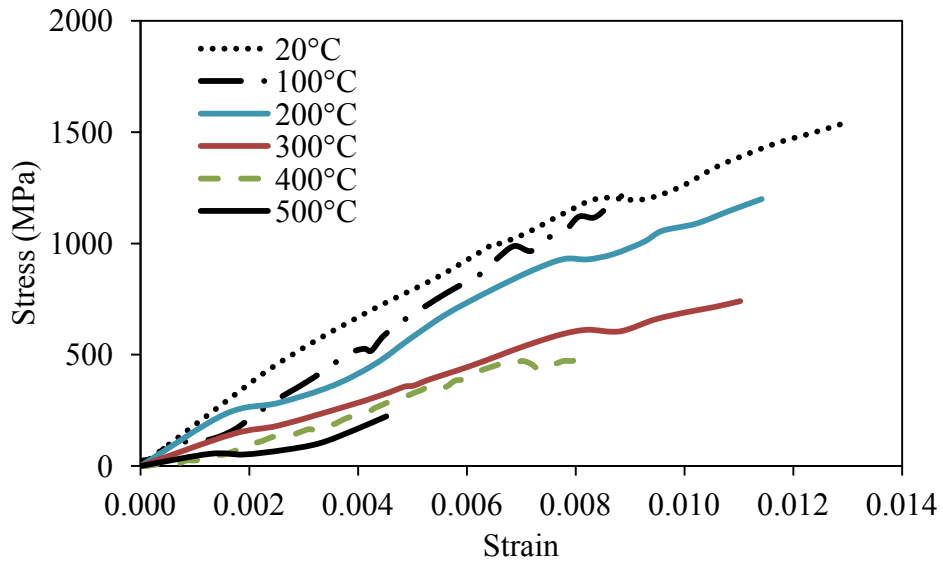


Figure 3.9 Stress-strain response of CFRP rods at various temperatures

The failure modes of CFRP strips and rods at various temperatures are illustrated in Figures 10 and 11. The failure pattern of CFRP specimens in 20-300°C range are quite similar, wherein CFRP splits into bunches of thin fibers due to cracking of polymer resin. These fibers then gradually are stretched or pulled out, and eventually CFRP specimen

loses its integrity as well as strength. Beyond 300°C, polymer resin starts to decompose, and carbon fibers also oxidize at temperatures above 400°C. It can be seen in Figures 10 and 11 that the fibers get more softened and separate out in 400-600°C temperature range. In these tests, the specimens eventually failed due to stretching of fibers at the mid-height.



Figure 3.10 Failure modes of CFRP strips at various temperatures



Figure 3.11 Failure modes of CFRP rods at various temperatures

3.2.4 Relations for tensile strength and modulus with temperature

Data generated from the above tests is utilized to develop empirical relations for strength and modulus of NSM CFRP reinforcement as a function of temperature. These relations are expressed in terms of temperature dependant reduction factors, which are normalized to room temperature values. A review of literature shows that there is very little information on degradation of mechanical properties of CFRP after resin decomposition. Mouritz and Gibson (2006) proposed the following general relation for the variation of mechanical properties of FRP with temperature.

$$P(T) = \left(\frac{P_U + P_R}{2} - \frac{P_U - P_R}{2} \tanh(k(T - T_g')) \right) R^n \quad (3.1)$$

$P(T)$ represents a particular property, either tensile or compressive strength, or elastic modulus; R^n is a power law factor to account the residual resin content. For tensile strength and elastic modulus, n can be considered to be zero, since tensile strength is mainly dependant on the strength of fibers after the decomposition of polymer resin, and thus $R^n = 1$. P_U and P_R are unrelaxed (low temperature) and relaxed (high temperature) values of that property, respectively. T_g' is the critical temperature of FRP, corresponding to a 50% reduction in the property value. k is a constant describing the extent of relaxation. This relation takes into account the effect of decomposition of FRP occurring at high temperatures on mechanical properties, and thus can be used over a wide range of temperatures. By dividing Eq. 3.1 by P_U , the retention factor for tensile strength and elastic modulus at a given temperatures can be obtained

$$F(T) = \frac{1 + P_R / P_U}{2} - \frac{1 - P_R / P_U}{2} \tanh(k(T - T_g')) \quad (3.2)$$

$F(T)$ is the retention (%) factor of mechanical properties at temperature T (°C).

The above equation is taken as the basis for developing an expression for strength and modulus retention factors for NSM CFRP. As discussed above, the resin of CFRP strips and rods gets completely evaporated at 600°C. Therefore, the strength and modulus at 600°C were used as P_R , and the strength and modulus at room temperature (20°C) were used as P_U . The critical temperature (T_g') corresponding to 50% reduction in tensile strength and modulus of NSM CFRP strip is 305°C and 340°C respectively, and the

corresponding values for NSM CFRP rod are 330°C and 320°C respectively. Then k is the only coefficient to be determined in Eq. 3.2.

A regression analysis was carried out using “Solver” function in Excel (2010) to determine k . The “Solver” is an advanced program in Excel which is able to obtain an optimum function to match a specified dataset. The prerequisite for using this Solver function is to provide a basic format of a function and coefficients to be determined. In the current analysis, Eq. 3.2 is the basic format of the function and k is the coefficient to be determined. Then a regression analysis was carried out so as to achieve a minimum error value between predictions from empirical formula (Eq. 3.2) and the above measured test data. Based on the regression analysis results, the following relations were arrived for strength and modulus retention factors in CFRP strip and rod as a function of temperature. CFRP strip:

$$\text{Strength: } f(T) = 0.56 - 0.44 \tanh(0.0052(T - 305)) \quad (3.3)$$

$$\text{Modulus: } E(T) = 0.51 - 0.49 \tanh(0.0035(T - 340)) \quad (3.4)$$

CFRP rod:

$$\text{Strength: } f(T) = 0.54 - 0.46 \tanh(0.0064(T - 330)) \quad (3.5)$$

$$\text{Modulus: } E(T) = 0.51 - 0.49 \tanh(0.0033(T - 320)) \quad (3.6)$$

The comparison of predicted strength and elastic modulus from proposed empirical relations with measured values in above discussed tests is plotted in Figures 3.12 and 3.13. It can be seen in Figure 3.12 that the proposed empirical relations closely match with measured data for tensile strength of CFRP strip and rod, and the average error between predicted strength and test data is 7% and 6.3% respectively. The elastic modulus predictions, as shown in Figure 3.13, also show reasonable agreement with test

data, and the average error is 10% and 11.2% for CFRP strips and rods respectively. This slight larger error in elastic modulus of CFRP strip and rod is mainly due to relatively scattered data obtained in tests.

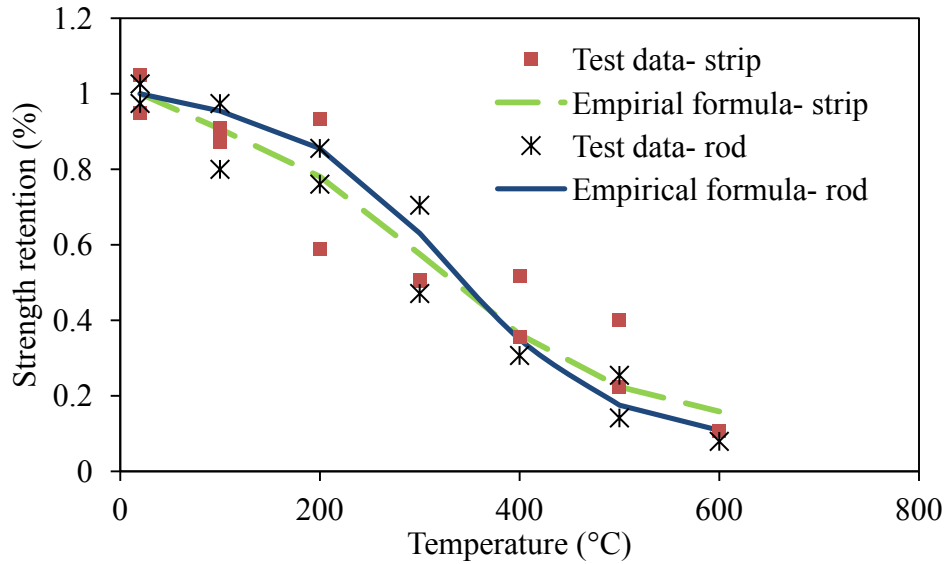


Figure 3.12 Comparison of tensile strength predicted by empirical formula with test data

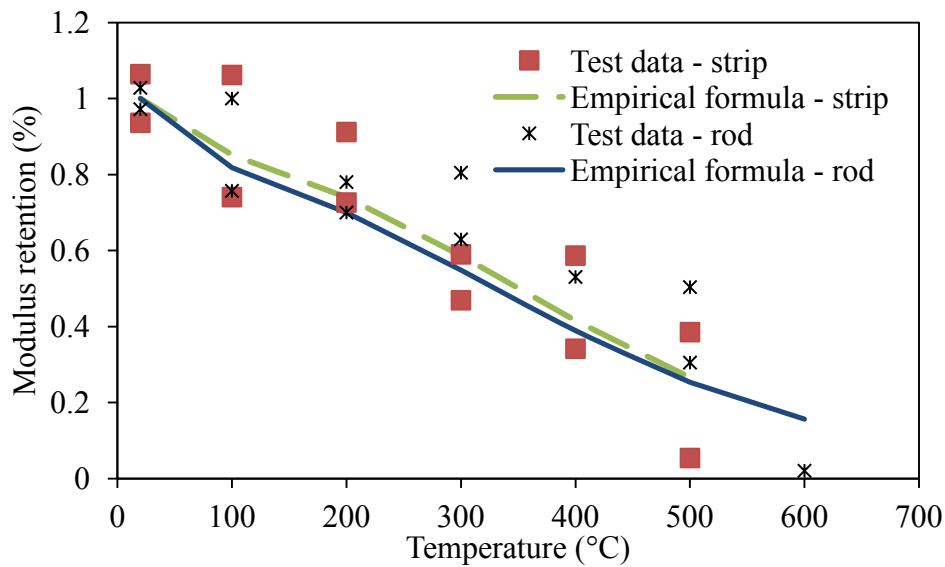


Figure 3.13 Comparison of elastic modulus predicted by empirical formula with test data

3.2.5 Summary of tension test results

Temperature has significant influence on tensile strength and stiffness properties of NSM CFRP reinforcement. NSM CFRP strips and rods retain much of their tensile strength and modulus till about 200°C. This is mainly due to the fact that polymer resin of CFRP remains intact up to 200°C. However, beyond 300°C, tensile strength and elastic modulus of NSM CFRP decrease at a faster pace due to decomposition of polymer resin. At 600°C, NSM CFRP only retains about 10% of its original strength.

CFRP strips and rods exhibit linear stress-strain response at both ambient and high temperatures. However, ultimate (failure) strain of CFRP decreases with increase in temperature, which is contrary to that occurring in reinforcing steel. NSM CFRP strips and rods exhibit slightly better resistance to high temperature as compared to conventional CFRP rebars and laminates. At last, empirical relations for strength and elastic modulus of CFRP strip and rod is proposed over a wide temperature range. These relations can be used in evaluating fire response of concrete structures strengthened with NSM FRP reinforcement.

3.3 Bond Strength Tests

This section presents results from an experimental study on the effect of temperature on bond strength and modulus of near-surface mounted (NSM) fiber-reinforced polymer (FRP) strengthened concrete. A series of NSM FRP specimens, fabricated using different types of epoxy adhesive and FRP reinforcement, were tested to evaluate bond strength in 20-400°C temperature range. Details of test procedure and results are presented as follows.

3.3.1 Preparation of test specimens

The experimental program consisted of 36 pull-out tests on NSM FRP specimens at various temperatures, as shown in Table 3.4. The NSM FRP specimens were made with two cross sectional shapes of CFRP types, strip and rod, embedded in two types of adhesives, namely Tyfo S epoxy and Tyfo T300 epoxy. The specimen preparation for bond strength tests comprised of three steps; casting of concrete block (Figure 3.14(a)), fabrication of FRP anchor (Figures 3.14(b) and 3.14(c)), and then fabrication of NSM FRP system (Figure 3.14(d)). The concrete blocks, of 150×150×400 mm size, were cast from a batch of pre-mixed concrete. The concrete mix comprised of Type I Portland cement, sand and carbonate based coarse aggregate. The measured compressive strength of concrete was 48 MPa on 28th day, and reached 50 MPa on 90th day.

Table 3.4 Bond test program on NSM FRP system

Test groups	NSM adhesive	Reinforcement	Temperature (°C)
I	Tyfo T300	CFRP strip (4x13.5 mm)	20-400
II	Tyfo T300	CFRP rod (dia. 6mm)	20-300
III	Tyfo S	CFRP strip (4x13.5 mm)	20-400
IV	Tyfo S	CFRP rod (dia. 6mm)	20-300



(a) Concrete block (b) Steel frame for anchor (c) Filling cement (d) Test specimen

Figure 3.14 Fabrication of NSM FRP bond test specimen

The concrete blocks were strengthened with two shapes of NSM FRP reinforcement, CFRP strips and CFRP rods. CFRP strips were of 4.5 mm thickness and 13.5 mm width, and CFRP rods were of 6.4 mm diameter. The nominal tensile strength and modulus of CFRP strip, as specified by the manufacturer, are 2790 MPa and 155 GPa respectively. While the nominal tensile strength and modulus of CFRP rod are 2070 MPa and 124 GPa respectively. The high strengths of CFRP strips and rods ensure that bond failure occurs prior to rupture of CFRP reinforcement. During preparation of NSM FRP specimens, one end of CFRP strip (or rod) was bonded to concrete blocks, and the other end was installed with a strong anchor, to facilitate gripping of CFRP strip (or rod) in the pull-out test. The anchor system was developed as per ACI 440.3 specifications (2004) and those recommended by Wang et al. (2007). This is achieved through filling expansive cement (RockFrac NEDA) into a circular steel tube (confinement), as shown in Figures 3.14(b) and 3.14(c).

After preparing concrete blocks and FRP anchors, concrete blocks were strengthened with NSM CFRP strips or rods. For this a groove was cut on the surface of casted concrete block. ACI 440.2 (2008) specifications recommend the groove size to be a minimum of 1.5 times the diameter of FRP rod. For FRP strips, the groove size needs to be at least $3.0a_b \times 1.5b_b$, where a_b is the smallest bar dimension and b_b is the length of the other edge, as shown in Figure 3.15. Therefore, two types of groove sections were cut on the surface of concrete blocks utilizing an electric saw. A groove size of 10×25 mm was cut for placing CFRP strips (4.5×13.5 mm in section), while a groove size of 13×13 mm was cut for CFRP round bars (6.4 mm in diameter). The bond length was set to be 150 mm for all specimens.

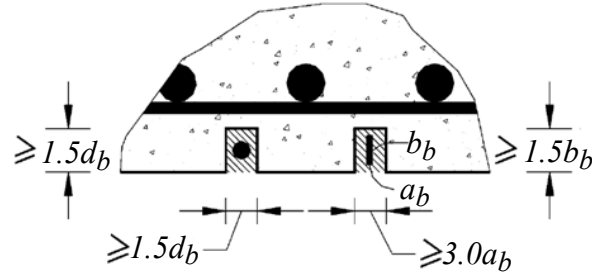


Figure 3.15 Groove size for installation of NSM FRP specified in ACI 440.2 (2008)

Two types of epoxy based adhesive, Tyfo S epoxy and Tyfo T300 epoxy, were used as groove fillers (adhesive). Tyfo S epoxy is a two-component matrix material and is marketed by FYFE Co. LLC. Tyfo S epoxy is recommended for its excellent bond properties at ambient conditions, and is widely used in externally bonded FRP strengthening systems. The specified glass transition temperature of Tyfo S epoxy is 82°C, and hence it might not exhibit good bond performance at elevated temperatures. An improvement over Tyfo S epoxy is Tyfo T300 epoxy, which has a higher glass transition temperature of 120°C. Thus Tyfo T300 epoxy might exhibit better bond behavior in fire resistance applications, but this is not quantified to date.

The concrete blocks were strengthened with CFRP strips or rods following the recommendations of Hughes Brothers, Inc. (2011). The grooves cut in concrete blocks were first filled with epoxy approximately till half depth. Then NSM FRP strip or rod was centered and inserted into the groove. Finally, the remaining space of the groove was filled with epoxy. The epoxy was allowed to cure for at least seven days before undertaking bond strength tests. A fabricated NSM FRP strengthened test specimen is shown in Figure 3.14(d).

3.3.2 Test set-up

For undertaking high temperature bond tests, a specialized set-up was designed and the test set-up is shown in Figure 3.16. The test equipment comprises of tension testing machine and an electric furnace to generate high temperature. In the tension testing machine, one end of the specimen, the concrete block, is held by a steel cage, which is connected to the top beam. The other end of the specimen, FRP with anchor system, is clipped to a pair of clamping brackets which are connected to the bottom beam. The bond specimen is loaded in tension by adjusting the distance between the top and the bottom beams. Two hydraulic jacks, sitting on the bottom steel beam, can directly apply specified load to the top beam through an extension rod. When hydraulic jacks apply an increasing load, the top beam moves upward and thus tensile force is applied on the NSM FRP specimen. During the test, the top beam is always maintained in a perfectly horizontal position to minimize onset of eccentric loading during the movement. The heating device comprises of a small scale electric furnace which can heat the entire steel cage and concrete block. Through this set-up, bond strength test can be conducted by heating NSM FRP system to a desired temperature and then subjecting it to tensile loading.

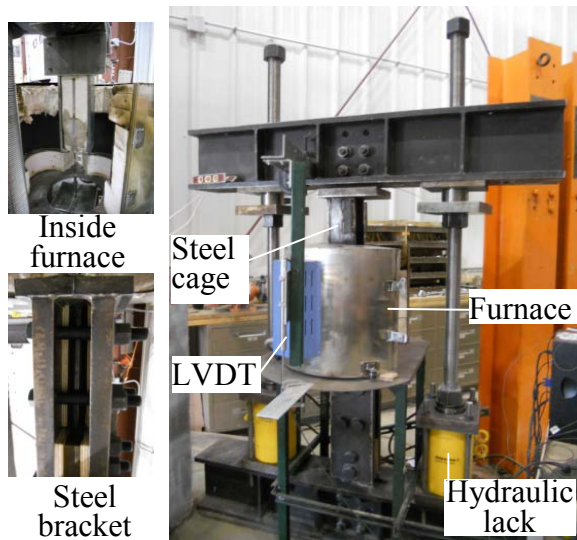


Figure 3.16 Test set-up for evaluating bond strength of NSM systems at high temperatures

During the test, NSM FRP specimen is heated to a target temperature, and then the heating is continued for additional 20 to 30 minutes to ensure the test specimen and furnace reach thermal equilibrium conditions. To accurately monitor the temperature difference between inside and outside of NSM epoxy, two thermocouples are embedded into NSM groove as well as on the surface of concrete block. The heating rate in the furnace is set to be at 5-10°C/min, depending on the target temperature: a higher heating rate is used for higher target temperatures.

Following the specimen attaining a target temperature, a pull-out test is carried out through the application of load using hydraulic jacks. To measure slip that occurs during a pull-out test, a linear variable differential transformer (LVDT) is placed between the top beam and the clamping brackets. The variation of distance between them is taken as the slip between CFRP reinforcement and concrete block, since the elongation of CFRP in anchor parts is negligible. The elongation measurements start as soon as loading

is applied, and the displacement of the top beam is recorded until CFRP strip or rod is pulled out.

3.3.3 Results and discussion

Data recorded in pull-out tests is utilized to evaluate bond strength and modulus of NSM FRP specimen at various temperatures. The bond strength (τ_{max}) and bond modulus (E) are evaluated using as:

$$\tau_{max} = P_{max} / A \quad (3.7)$$

$$E = \Delta\tau / \Delta\epsilon_{slip} \quad (3.8)$$

where P_{max} is the maximum load recorded in the tension test, A is the area of contact surface between CFRP and NSM epoxy, $\Delta\tau$ and $\Delta\epsilon_{slip}$ are the relative bond stress and slip strain on linear part of bond stress-strain curve.

The slip strain is evaluated as

$$\epsilon_{slip} = s / L_{bond} \quad (3.9)$$

where s is the slip measured in the test, and L_{bond} is the bond length of test specimen.

At each target temperature, two pull-out tests were carried out, and the average of two values was taken as bond strength and modulus of NSM FRP system. Results from these tests at various temperatures are tabulated in Tables 3.5 to 3.8 for CFRP strip and rod with Tyfo T300 and Tyfo S epoxy respectively.

3.3.3.1 Bond strength and modulus at room temperature

The bond strengths of NSM CFRP specimens with Tyfo T300 epoxy at room temperature are found to be 7.04 and 10.33 MPa for CFRP strips and CFRP rods respectively, and the corresponding values with Tyfo S epoxy are 3.57 and 3.42 MPa respectively. The bond strength of CFRP strip and rod with Tyfo T300 epoxy is significantly higher than that casted with Tyfo S epoxy. The main reason for this difference can be attributed to different failure patterns that occurred in these two types of epoxy. In NSM CFRP specimens fabricated with Tyfo T300 epoxy, bond failure occurred at epoxy-concrete interface, and a thin layer of concrete got detached from concrete blocks with CFRP strip or CFRP rod. This indicates that Tyfo T300 epoxy possesses good adhesion with CFRP strip or CFRP rod in use, and this helps to develop a stronger bond at FRP-epoxy interface. Thus failure in this case is through progression of cracking in concrete and this leads to development of higher bond strength.

In contrast, in NSM FRP specimens fabricated with Tyfo S epoxy, failure occurred through pull-out of CFRP strips or rods, and the bond at epoxy-concrete interface was barely affected. This indicates that shear stress between CFRP and Tyfo S epoxy is relatively lower than those at concrete-epoxy interface, and thus failure occurs through debonding between CFRP and epoxy. A comparison of bond modulus, evaluated for NSM CFRP with two types of adhesive, also indicates that Tyfo T300 possess higher bond modulus than that of Tyfo S epoxy.

Table 3.5 Bond strength and modulus of Tyfo T300 epoxy
for NSM CFRP strip at various temperatures

Temp. (°C)	Force (kN)	Bond strength (MPa)	Avg. bond strength (MPa)	% of initial bond strength	Bond modulus (MPa)	Avg. bond modulus (MPa)	% of initial bond modulus	Failure mode
20	45.12	8.22	7.04	100.0	227.32	181.07	100.0	Fracture of concrete edge or cracking of epoxy
	32.08	5.85			134.81			
100	18.25	3.33	3.94	56.0	127.22	119.01	65.7	Debonding at bar-epoxy interface
	25.01	4.56			110.80			
200	8.53	1.56	1.35	19.3	42.60	44.95	24.8	Debonding at bar-epoxy interface
	6.33	1.15			47.29			
300	6.62	1.21	1.15	16.3	20.72	29.15	16.1	Debonding at bar-epoxy interface
	6.00	1.09			37.57			
400	2.96	0.54	0.65	9.3	11.97	13.97	7.7	Debonding at bar-epoxy interface
	4.21	0.77			15.97			

Table 3.6 Bond strength and modulus of Tyfo T300 epoxy
for NSM CFRP rod at various temperatures

Temp. (°C)	Force (kN)	Bond strength (MPa)	Avg. bond strength (MPa)	% of initial bond strength	Bond modulus (MPa)	Avg. bond modulus (MPa)	% of initial bond modulus	Failure mode
20	34.15	11.15	10.33	100.0	197.75	186.54	100.0	Fracture of concrete edge or cracking of epoxy
	29.13	9.51			175.32			
100	24.29	7.93	7.09	68.7	154.02	142.56	76.4	Debonding at bar-epoxy interface
	19.16	6.25			131.09			
200	7.09	2.31	3.18	30.8	43.45	43.65	24.0	Debonding at bar-epoxy interface
	12.41	4.05			43.84			
300	4.10	1.34	1.24	12.0	39.69	32.67	17.9	Debonding at bar-epoxy interface
	3.49	1.14			25.65			

Table 3.7 Bond strength and modulus of Tyfo S epoxy
for NSM CFRP strip at various temperatures

Temp. (°C)	Force (kN)	Bond strength (MPa)	Avg. bond strength (MPa)	% of initial bond strength	Bond modulus (MPa)	Avg. bond modulus (MPa)	% of initial bond modulus	Failure mode
20	18.38	3.35	3.57	100.0	93.49	90.87	100.0	Debonding at bar-epoxy interface
	20.83	3.80			88.26			
100	8.58	1.56	2.24	62.8	55.84	62.67	69.0	Debonding at bar-epoxy interface
	16.04	2.92			69.49			
200	4.81	0.88	1.07	29.9	37.54	39.55	43.5	Debonding at bar-epoxy interface
	6.92	1.26			41.56			
300	3.64	0.66	0.81	22.7	44.18	31.79	35.0	Debonding at bar-epoxy interface
	5.27	0.96			19.40			
400	4.03	0.73	0.67	18.8	27.55	23.19	25.5	Debonding at bar-epoxy interface
	3.33	0.61			18.83			

Table 3.8 Bond strength and modulus of Tyfo S epoxy
for NSM CFRP rod at various temperatures

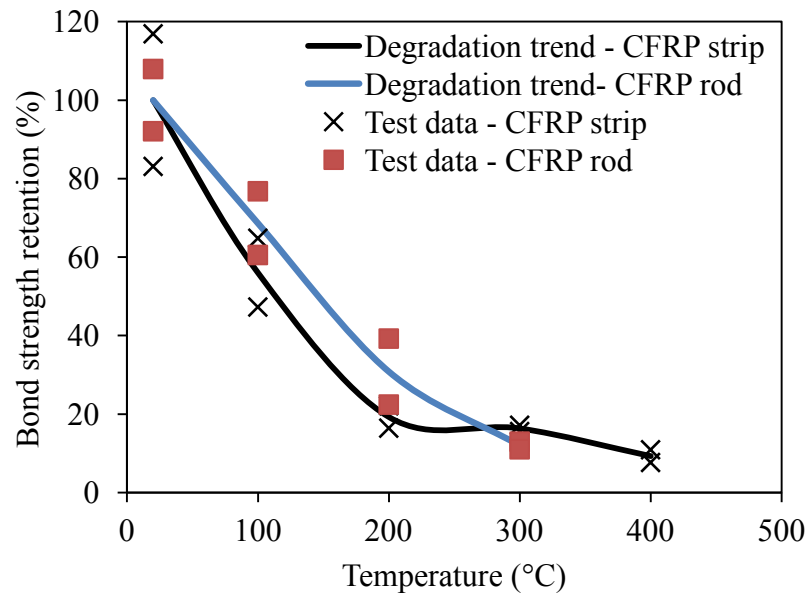
Temp. (°C)	Force (kN)	Bond strength (MPa)	Avg. bond strength (MPa)	% of initial bond strength	Bond modulus (MPa)	Avg. bond modulus (MPa)	% of initial bond modulus	Failure mode
20	11.10	3.62	3.42	100.0	76.72	71.17	100.0	Debonding at bar-epoxy interface
	9.87	3.22			65.61			
100	4.68	1.53	1.94	56.6	43.35	54.80	77.0	Debonding at bar-epoxy interface
	7.19	2.35			66.24			
200	4.03	1.32	1.13	33.0	30.45	28.18	39.6	Debonding at bar-epoxy interface
	2.88	0.94			25.91			
300	1.46	0.48	0.61	17.8	13.86	16.04	22.5	Debonding at bar-epoxy interface
	2.27	0.74			18.21			

3.3.3.2 Bond strength and modulus at elevated temperature

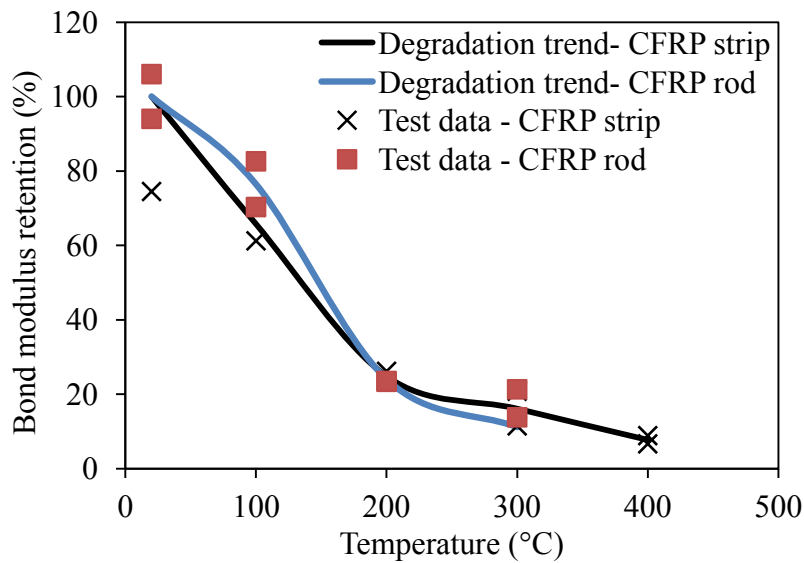
The bond strength and modulus of NSM FRP strengthening system at elevated temperatures were evaluated using measured failure load and displacement. The variation of bond strength and bond modulus of NSM FRP is shown in Figures 3.17 and 3.18, by plotting normalized bond strength and modulus as a function of temperature. It can be seen that in both cases of Tyfo T300 and Tyfo S epoxy, bond strength and modulus of NSM strengthening system degrade quickly with increasing temperature, and this degradation can be grouped into two stages. In 20-200°C temperature range, bond strength decreases at a relatively faster pace, and NSM FRP system only retains 20-30% of its original bond strength at 200°C. This rapid deterioration is mainly due to softening of epoxy beyond glass transition temperature (around 70°C), and thus the adhesion between FRP and epoxy gets degraded. Beyond 200°C, epoxy adhesive experiences melting and decomposition, and thus bond properties further deteriorate with temperature. Since NSM FRP system has already lost most of its bond strength and stiffness at around 200°C, the rate of degradation at this stage is relatively low. Observations during bond tests indicate that epoxy starts to burn at around 400°C and this damages NSM bond. Thus bond strength becomes negligible at 400°C for CFRP strips and 300°C for CFRP rods. Therefore, no further tests were conducted beyond these temperature levels.

A comparison of bond test data plotted in Figures 3.17(a) and 3.18(a) indicates that CFRP rods possess slightly higher bond strength than those of CFRP strips. This can be attributed to the fact that CFRP rod is embedded in concrete block on all surfaces, and this helps to develop higher confinement in epoxy adhesive. However, for the same type of epoxy, measured bond forces are very close for CFRP strips and CFRP rods (see

Tables 2-5). This indicates that at high temperatures, shape of FRP reinforcement (strip or rod) does not significantly influence bond properties of NSM FRP system.

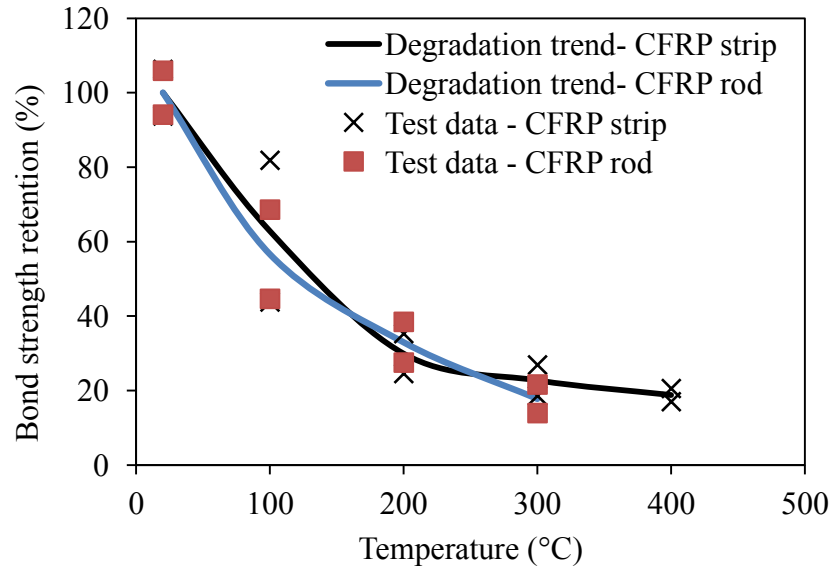


(a) Bond strength

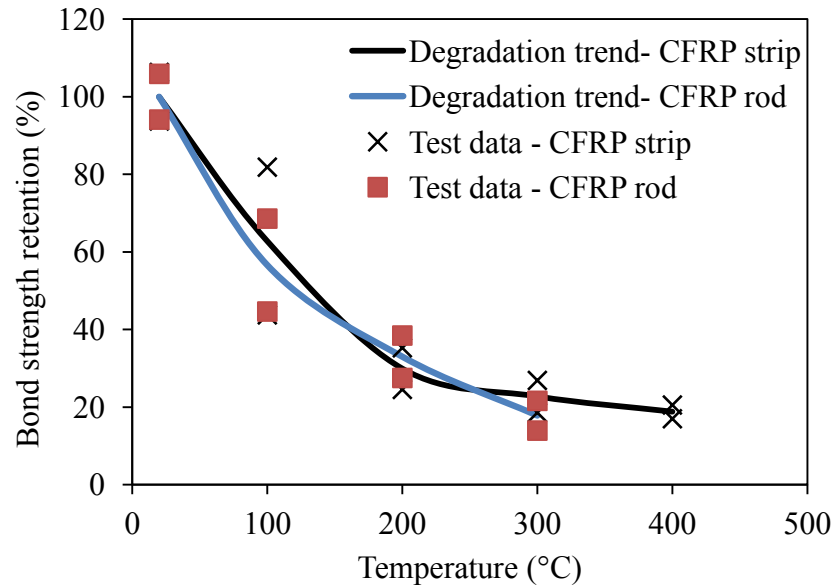


b. Bond modulus

Figure 3.17 Variation of bond strength and elastic modulus of NSM CFRP strip and rod with Tyfo T300 epoxy with temperature



(a) Bond strength

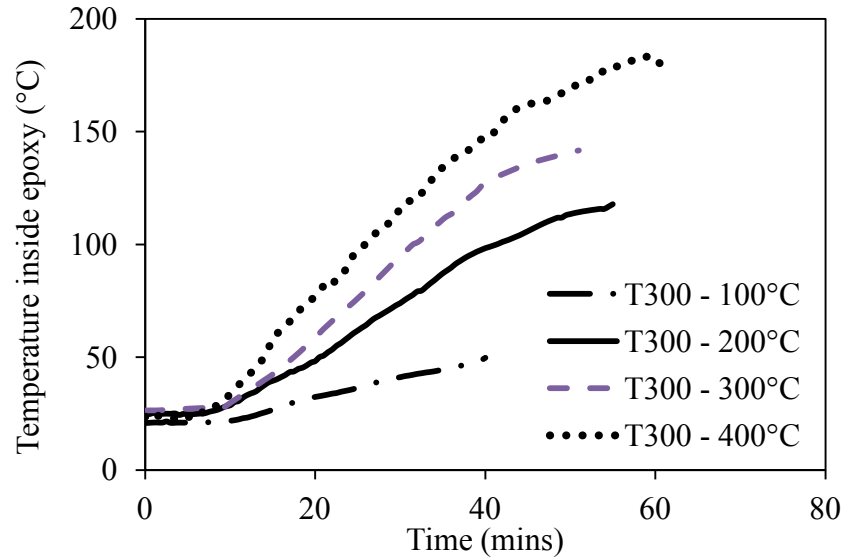


(b) Bond modulus

Figure 3.18 Variation of bond strength and bond modulus of NSM CFRP strip and rod with Tyfo S epoxy with temperature

A review of trends plotted in Figures 3.17 and 3.18 infer that NSM CFRP with Tyfo T300 epoxy possesses higher bond strength and modulus than those of Tyfo S

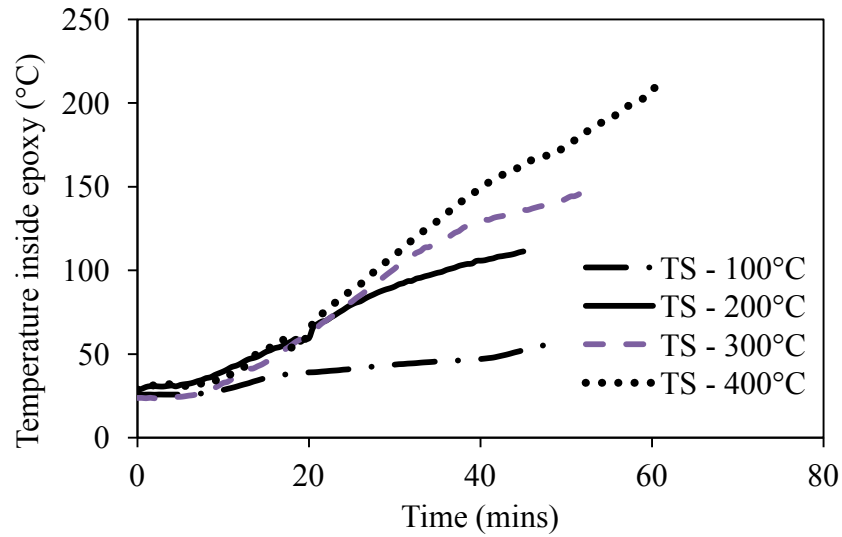
epoxy at elevated temperatures. This higher bond strength in NSM CFRP with Tyfo T300 epoxy can be attributed to better thermal insulation effect facilitated by Tyfo T300 epoxy. As shown in Figure 3.19, temperature rise in Tyfo T300 epoxy is relatively lower than that in Tyfo S epoxy, and this mainly results from higher glass transition temperature of Tyfo T300 epoxy. Thus, higher retention of bond strength is achieved NSM CFRP specimens with Tyfo T300 epoxy. Also, Tyfo T300 epoxy possesses relatively better adhesion with CFRP strips or rods, as found in the ambient temperature tests. Thus at elevated temperatures, this better adhesion also helps to achieve higher bond strength.



(a) Tyfo T300 epoxy

Figure 3.19 Variation of temperature inside Tyfo T300 and Tyfo S epoxy as a function of heating time

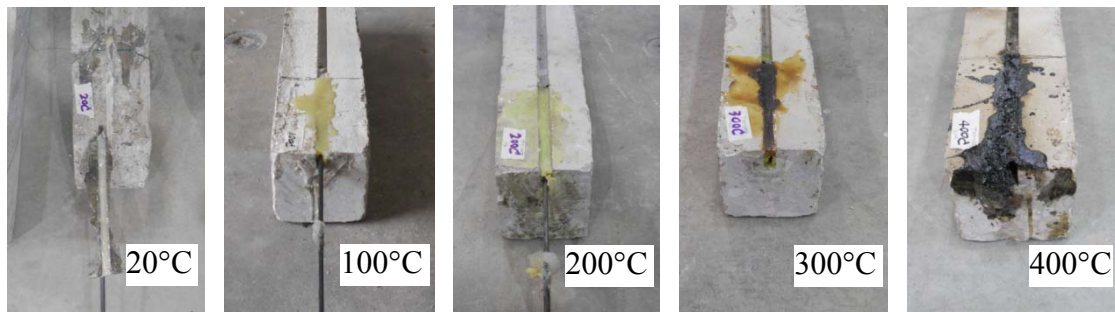
Figure 3.19 (cont'd)



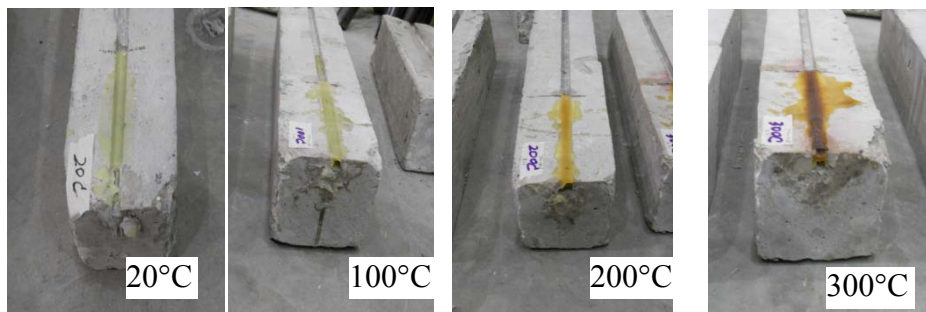
(b) Tyfo S epoxy

The failure mode of NSM CFRP specimens with Tyfo T300 and Tyfo S epoxy in high temperature pull-out tests are shown in Figures 3.20 and 3.21 respectively. It can be seen that all NSM CFRP specimens failed through debonding at FRP-epoxy interface at high temperatures, and both CFRP strips and rods were pulled out from NSM adhesive. This failure mode is in contrast to that experienced at room temperature, and this is also an indicator of lower bond strength in NSM FRP system at elevated temperatures. As can be seen in Figures 3.20 and 3.21, at 100°C and 200°C, CFRP strips or rods were directly pulled out, and there was no obvious damage either in NSM epoxy or in concrete block. This indicates that the NSM epoxy gets softened, leading to significant decrease in the shear resistance at CFRP-epoxy interface. Further, in tests at 300°C, the color of epoxy turned black, and this infers that epoxy underwent chemical reaction (charring) and started decomposing. When temperature raised to 400°C, epoxy experienced significant pyrolysis, and NSM CFRP system was severely damaged as shown in Figures 3.20 and

3.21. Thus, no bond (strength) was left in NSM CFRP system at temperatures beyond 400°C.

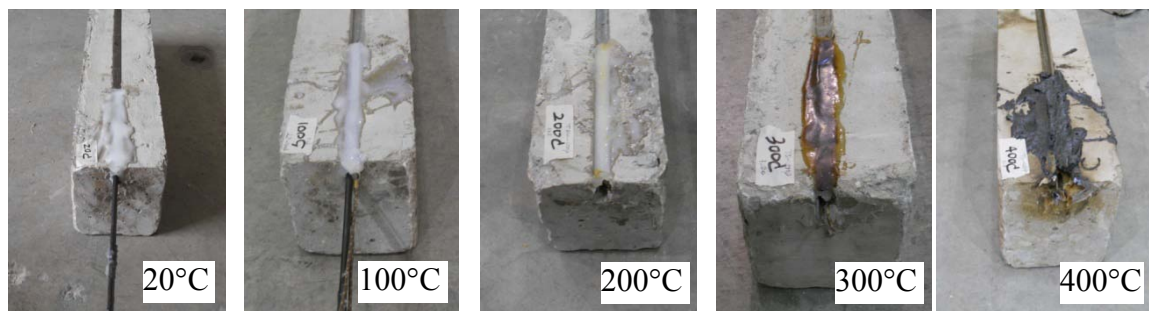


(a) CFRP strips

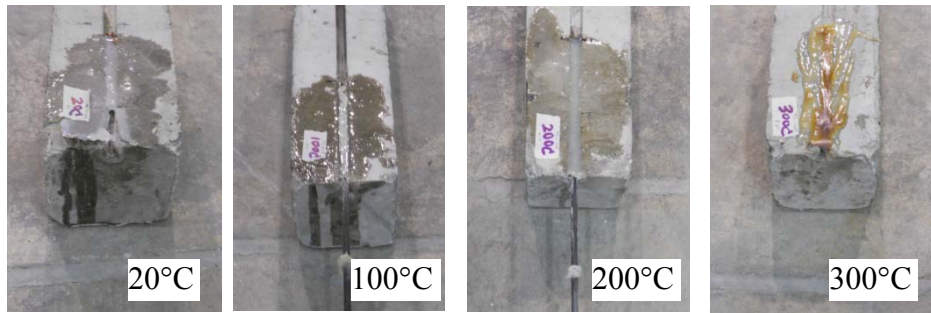


(b) CFRP rods

Figure 3.20 Failure modes of NSM CFRP specimens with Tyfo T300 epoxy



(a) CFRP strips



(b) CFRP rods

Figure 3.21 Failure modes of NSM CFRP specimens with Tyfo S epoxy

3.3.3.3 Bond stress-slip relations

The bond stress-slip relationships for NSM CFRP system, with Tyfo T300 and Tyfo S epoxy, are shown in Figures 3.22 and 3.23 respectively. It can be seen that all NSM CFRP systems exhibit similar stress-slip response in pull-out tests, regardless of epoxy type and reinforcement type. The measured bond stress-slip response can be grouped under two distinct stages: pre-peak stage and post-peak stage. In pre-peak stage, the bond stress increases at a high rate and quickly reaches its peak value, and the slip between CFRP and concrete is quite small. In this stage, there is good adhesion between CFRP and epoxy adhesive, and the measured slip is roughly equivalent to elastic deformation of CFRP and epoxy adhesive.

Past the peak point (post-peak stage), the bond stress deteriorates, and this is mainly due to onset of cracking in epoxy adhesive. In this stage, NSM system might regain some of its lost bond strength, and then bond strength gradually decreases until FRP is pulled out. This gradual decrease can be attributed to interlock action between CFRP and epoxy adhesive, and this interlock action remains effective until FRP is totally pulled out.

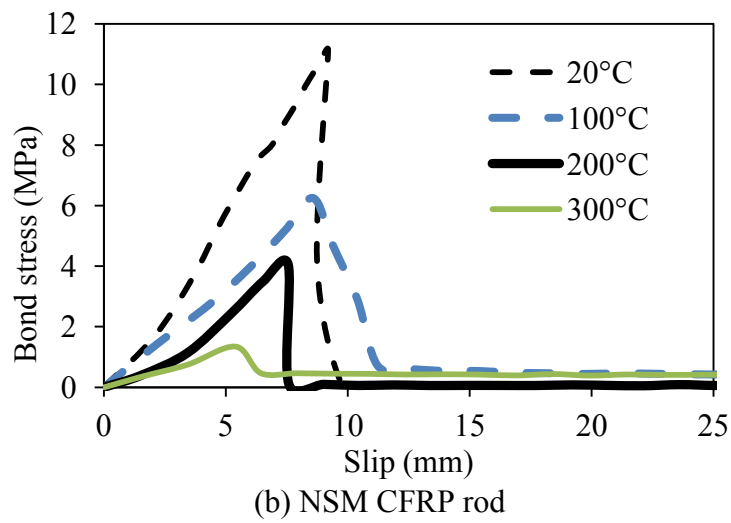
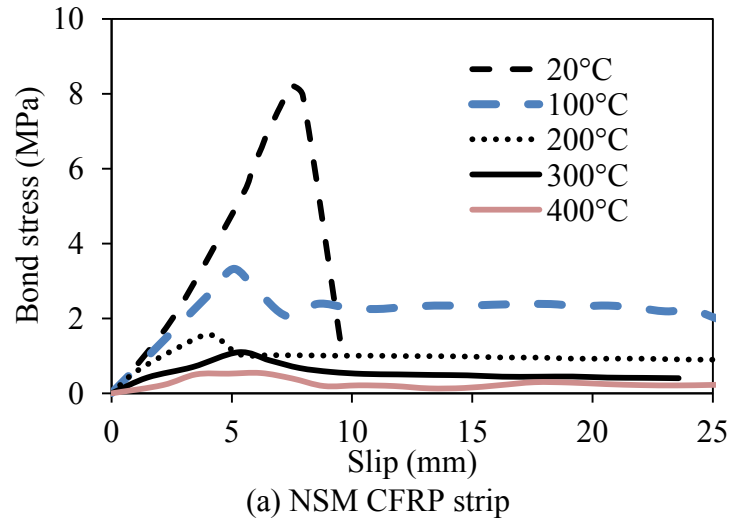
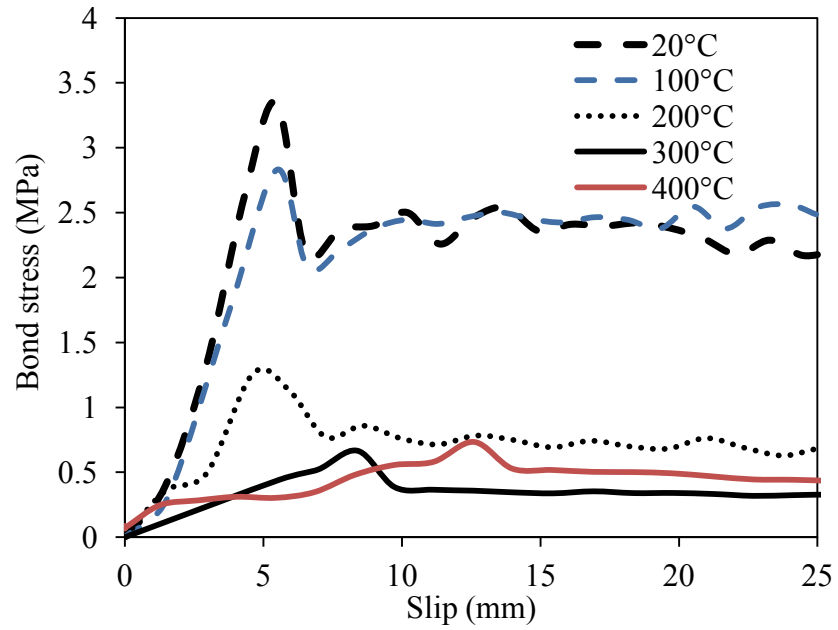
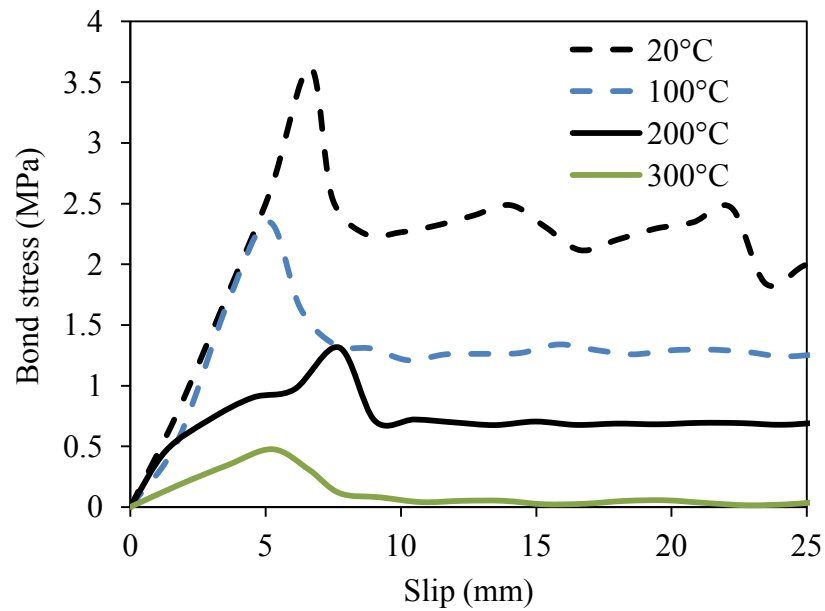


Figure 3.22 Bond stress-slip relations for NSM CFRP specimens with Tyfo T300 epoxy at various temperatures



(a) NSM CFRP strip



(b) NSM CFRP rod

Figure 3.23 Bond stress-slip relations for NSM CFRP specimens with Tyfo S epoxy at various temperatures

It can be seen from Figures 3.22 to 3.23 that the bond stress-slip responses for both CFRP strips and CFRP rods almost follow the same trend at various temperatures. However, at elevated temperatures, both bond stress and bond modulus decrease significantly, and thus the ascending phase and descending phase of bond stress-slip curves are not obvious at high temperatures. This also leads to more evenly distributed bond stress along the bond length.

3.3.4 Relations for bond strength and modulus with temperature

Data generated from the above tests is utilized to develop empirical relations for bond strength and bond modulus of NSM CFRP system as a function of temperature. The above test results indicated that NSM adhesive (epoxy) is the primary factor influencing bond strength and modulus at various temperatures, and shape of CFRP reinforcement does not have significant influence on the degradation of bond properties. Thus empirical relations were developed for NSM CFRP reinforcement with Tyfo T300 epoxy and Tyfo S epoxy respectively, and these relations are expressed in terms of temperature dependant reduction factors. These reduction factors of high temperature bond strength and modulus are normalized to their corresponding room temperature values.

So far no bond strength-temperature relations are available for NSM FRP strengthening system. A review of literature indicated that the following relations for bond degradation in concrete member reinforced with internal FRP rebars is available (Katz and Berman 2000, Bisby et al. 2005).

$$\tau(T) = 0.5(1 - \tau_r) \tanh(k(T - a)) + 0.5(1 + \tau_r) \quad (3.10)$$

where $\tau(T)$ represents normalized bond strength at temperature T , τ_r represents normalized residual bond strength. k is a constant related to degree of cross-link of polymer epoxy. a is a constant related to glass transition temperature of polymer epoxy.

Bond degradation in concrete with internal FRP rebars mainly results from temperature induced softening of FRP epoxy, which is similar to debonding mechanism of NSM FRP system based on the observations and data obtained from this test program. Thus, the above proposed relation (Eq. 3.10) for bond degradation in concrete with internal FRP rebars is modified to account for NSM CFRP system.

A regression analysis was performed using “Solver” function in Excel (2010) for developing modified expressions for bond strength and modulus retention factors of NSM FRP system. The “Solver” is an advanced program in Excel which is able to obtain an optimum function to match a specified dataset. The prerequisite for using this Solver function is to provide a basic format of a function and coefficients to be determined. The decreasing sigmoidal expression of Eq. 3.10 is taken as the basic format. The retention of bond strength and modulus at 400°C is used as τ_r , since NSM FRP systems lose most of their bond strength at that temperature. Thus k and a are the only coefficients to be determined in Eq. 3.10.

Then a regression analysis was carried out so as to achieve a minimum error value between predictions from empirical formula (Eq. 3.10) and the above measured test data. Based on the regression analysis results, the following temperature dependant relations were arrived at for bond strength and bond modulus retention factors of NSM CFRP with Tyfo T300 and Tyfo S epoxy respectively.

Tyfo T300 epoxy:

$$\text{Bond strength: } \tau(T) = 0.55 - 0.45 \tanh(0.011(T - 119)) \quad (3.11)$$

$$\text{Bond modulus: } E(T) = 0.59 - 0.41 \tanh(0.01(T - 143)) \quad (3.12)$$

Tyfo S epoxy:

$$\text{Bond strength: } \tau(T) = 0.55 - 0.45 \tanh(0.012(T - 129)) \quad (3.13)$$

$$\text{Bond modulus: } E(T) = 0.6 - 0.4 \tanh(0.009(T - 143)) \quad (3.14)$$

A comparison of predicted bond strength and modulus from empirical relations (Eq. 3.13 and Eq. 3.14) with measured values from above discussed tests is plotted in Figures 3.24 and 3.25. It can be seen in Figure 3.24 that the proposed empirical relations reasonably agree with measured data for bond strength of Tyfo T300 and Tyfo S epoxy, and the average error between predicted bond strength and test data is 6.7% and 8.2% respectively. The bond modulus predictions, as shown in Figure 3.25, also show good agreement with test data, and the average error is 6.3% and 6.8% for Tyfo T300 and Tyfo S epoxy respectively. It can be seen that Tyfo T300 epoxy exhibits slightly higher degradation in bond strength and bond modulus than those of Tyfo S epoxy. This is mainly attributed to the fact that Tyfo T300 epoxy possesses relatively higher bond strength at room temperature. However, in comparison to actual bond strength at various temperatures, Tyfo T300 epoxy exhibits better bond performance than Tyfo S epoxy.

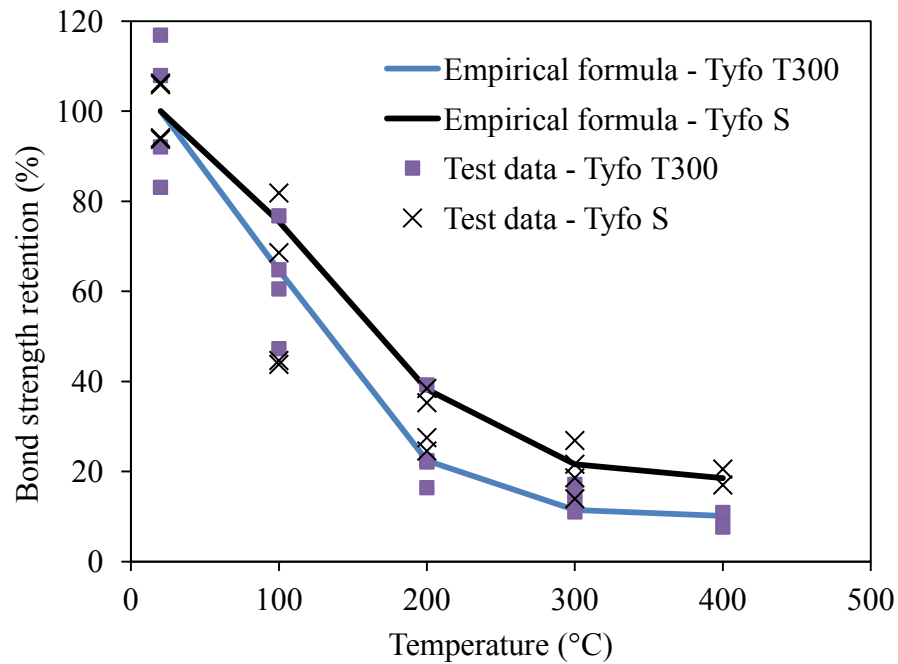


Figure 3.24 Comparison of predicted bond strength from proposed empirical relations with measured data from tests

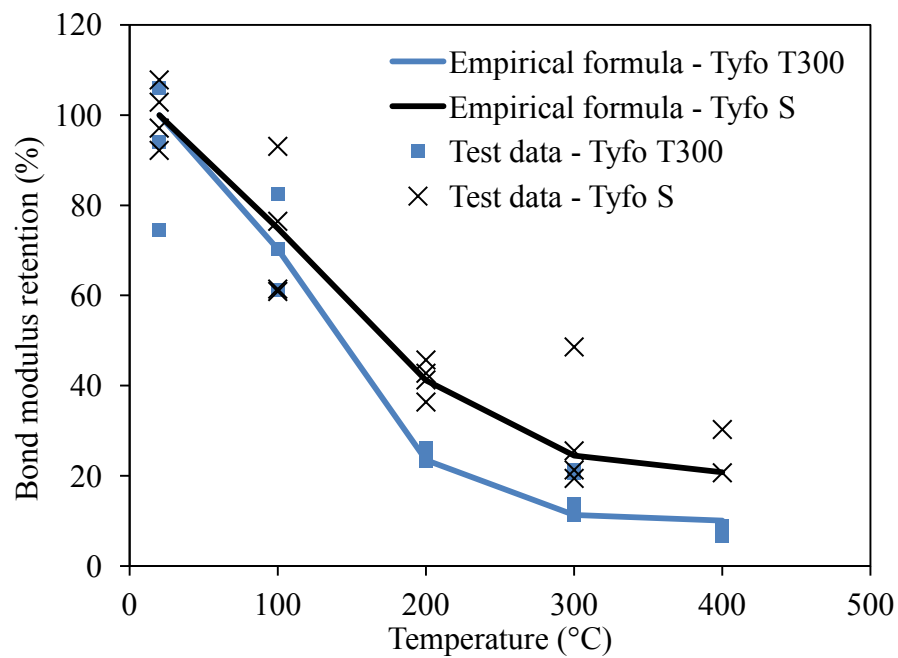


Figure 3.25 Comparison of predicted bond modulus from proposed empirical relations with measured data from tests

3.3.5 Summary of bond test results

Based on the above bond tests, bond strength of NSM FRP system are mainly dependant on the type of epoxy adhesive, rather than shape of FRP reinforcement (strip or rod). At elevated temperatures, the failure mode of NSM CFRP system with Tyfo T300 epoxy is through pull-out of CFRP strips or rods. This is contrast to room temperature failure mode, which is through detachment of concrete layer. The bond strength and modulus of NSM CFRP system decrease by about 80% of their original values at 200°C, and becomes negligible at 400°C. NSM CFRP system with Tyfo T300 epoxy exhibits higher bond strength and bond modulus than those of NSM CFRP system in the entire 20-400°C range.

Bond stress-slip response of NSM CFRP system exhibits two distinct stages: pre-peak stage and post-peak stage. Bond stress-slip responses at both room and high temperatures (in 20-400°C range) follow a similar pattern. However, the peak value (bond strength) and the slope (bond modulus) are lower at elevated temperatures.

At last, the proposed temperature dependant relations for degradation of bond strength and bond modulus of NSM CFRP system can be used for evaluating fire response of concrete structures strengthened with NSM CFRP reinforcement.

3.4 Thermal Expansion Tests

Thermal expansion of FRP varies in the longitudinal and transverse directions depending on the types of fiber, resin, and volume fraction of fiber. Previous studies on thermal expansion of FRP were mainly focused on internal FRP rebars, but no studies were conducted on thermal expansion of NSM FRP strips. This section provides detailed

test procedure and results on thermal expansion test of typical FRP reinforcement for NSM application.

3.4.1 Preparation of test specimens

To evaluate thermal expansion of FRP reinforcement at elevated temperatures, a set of thermal expansion tests were conducted utilizing Thermal Mechanical Analyzer (TMA). Four types of commercially available FRP reinforcement, provided by two manufacturers, are tested in this program. They are Aslan GFRP 100 rod, Aslan CFRP 200 rod, Tyfo CFRP strip and Tyfo CFRP rod. All these FRP products are used for NSM strengthening applications. The dimensions and properties of FRP samples in use are tabulated in Table 3.9.

Table 3.9 NSM FRP specimens used for thermal expansion test

FRP specimens	Dimensions (mm)		Fiber content (%)	T_g (°C)
	Section	Length		
Aslan GFRP 100	dia. 9	10/20	>70 (weight)	>110
Aslan CFRP 200	dia.13	10/20	N/A	>110
Tyfo CFRP strip	13.5×4.5	10/20	62 (volumetric)	71
Tyfo CFRP rebar	dia.6	10/20	60 (volumetric)	71

Based on ISO 11358 (1999) and ASTM E831 (2012) standard, the specimens used for thermal expansion test should be of 5-10 mm in length and width, and the two ends of test specimens should be parallel. Thus, FRP specimens were cut into around 10 mm in length, and the transverse dimension (width or diameter) was trimmed to be within 10 mm. To ensure reliability of measurements, thermal expansion tests are repeated at least once.

3.4.2 Test apparatus and test procedure

For thermal expansion measurements, thermo-mechanical analyzer (TMA) apparatus was used in the test, as shown in Figure 3.26. TMA utilizes a movable-core linear variable differential transducer (LVDT), which generates an output signal directly proportional to the specimen's dimension change. TMA can be used for measuring dimensional changes in a specimen from room temperature to 1000°C. A flat-tipped standard expansion probe is placed on the specimen, and a small static force is applied to it so that the probe stays on the specimen. The specimen is subjected to a temperature increase regiment according to a user-defined temperature ramp, and the probe movement records the sample expansion or contraction (Kodur and Khaliq 2011).

Before the test, FRP specimen is placed on a pedestal in the moveable furnace of the TMA and the expansion probe is set on the specimen, as shown in Figure 3.26. Once specimen is placed in position, the test can be run and controlled by computer, which records dimensional change with increasing temperatures. Based on the recommendation from TMA manufacturer (TA 2007), the heating rate of thermal expansion test was set to be 3°C/min. The temperature range in test was constrained to 20-300°C, since FRP starts to decompose beyond 300°C and then this might damage the test equipment.

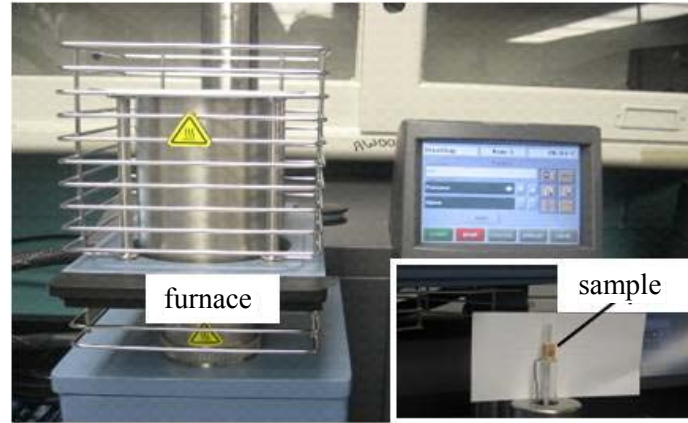
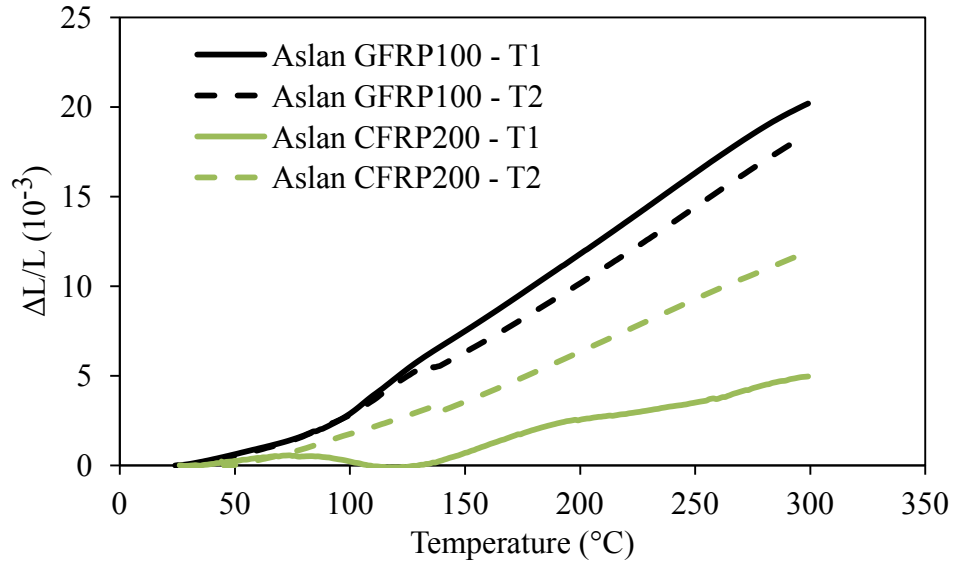


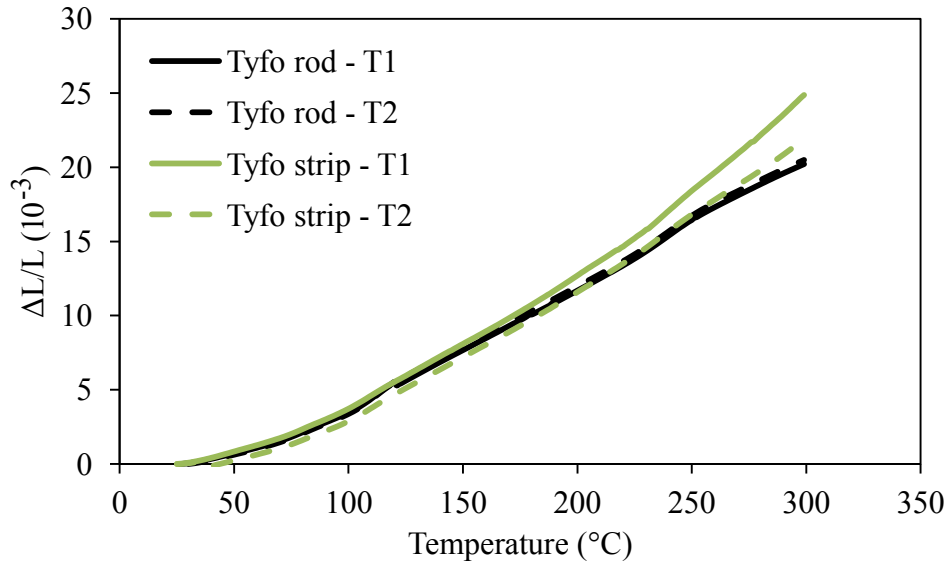
Figure 3.26 TMA apparatus and setup for thermal expansion test

3.4.3 Results and discussion

Measured dimensional changes of NSM FRP specimens were recorded to evaluate their thermal expansion in a wide temperature range. The variation of transverse dimension of FRP specimens is shown in Figure 3.27, by plotting a unit dimensional change as a function of temperature. It can be seen in Figures 3.27(a) and 3.27(b) that the transverse dimension of FRP keeps increasing at elevated temperatures, regardless of fiber type (glass or carbon) or cross section shape (strip or rod). This is mainly attributed to the fact that thermal expansion in transverse direction is dominated by the properties of polymer matrix of FRP, and polymer expands significantly at elevated temperatures. As shown in Figure 3.27(a), Aslan GFRP exhibits a relatively larger thermal expansion than that of Aslan CFRP, which might result from more sensitive response of glass fibers to thermal effect as compared to carbon fibers. For Tyfo rods and strips investigated, their thermal expansion responses were very similar throughout the tests. This is mainly due to similar fiber content (61%) and transverse dimensions (6 mm for rods and 4.5 mm for strips) of these specimens.



(a) Thermal expansion of Aslan GFRP and Aslan CFRP

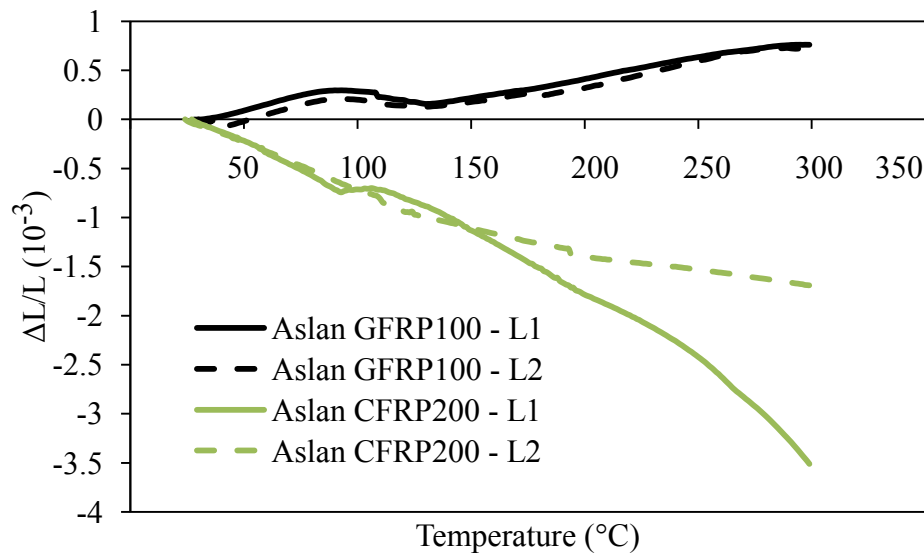


(b) Thermal expansion of Tyfo rods and Tyfo strips

Figure 3.27 Thermal expansion of NSM FRP specimens in transverse directions

Unlike thermal response in transverse direction, the variations of NSM FRP in longitudinal direction are relatively small. It can be seen in Figures 3.28(a) and 3.28(b), the dimensional changes in longitudinal direction are mostly negative values for CFRP

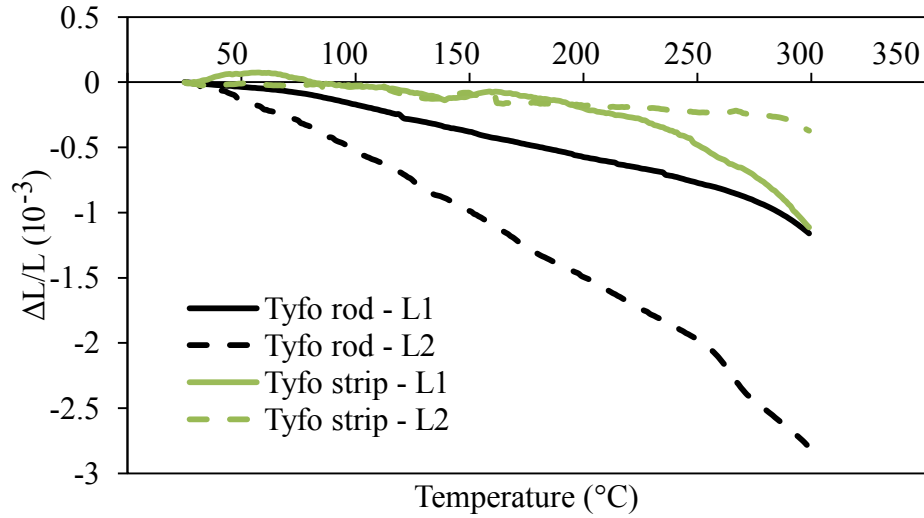
specimens, which indicates that CFRP actually experiences shrinking at elevated temperatures. For Aslan GFRP specimen, it still expands in longitudinal direction at elevated temperature, but the elongation per unit length gets significantly decreased. This lower expansion is due to the fact that longitudinal thermal expansion is dominated by the properties of fibers in FRP. Fibers, especially carbon fibers, usually have very small thermal deformation (Bank 1993). This leads to negligible thermal expansion of FRP composite in longitudinal direction. Since the epoxy of FRP gets softened at elevated temperatures, FRP specimens can easily buckle in longitudinal direction. Thus the longitudinal thermal expansion data usually varies considerably in the test. The test results plotted in Figure 3.28 indicate that longitudinal dimensional change of FRP fluctuated around $(-3 \sim 1) \times 10^{-3}$ per unit length.



(a) Thermal expansion of Aslan GFRP and Aslan CFRP

Figure 3.28 Thermal expansion of NSM FRP specimens in longitudinal directions

Figure 3.28 (cont'd)



(b) Thermal expansion of Tyfo rods and Tyfo strips

Coefficient of thermal expansion (CTE) is a most widely used parameter for accessing thermal elongation of materials. Based on the recommendation from ISO 11358 standards (1999), the coefficient of thermal expansion is calculated using the following equation:

$$\alpha = \frac{dL}{dT} \times \frac{1}{L_0} \quad (5.1)$$

where L_0 is the sample length at room temperature, dL is the change in length at temperature T , and dT is the change in temperature. ACI 440.1 specification (2006) provides a set of CTE values for various types of FRP. However, these data was generated in a limited temperature range, and they were mainly for internal reinforcing bars. Thus, the data generated in this test was analyzed to compute CTE over a wide temperature range for NSM FRP reinforcement, as tabulated in Table 3.10.

It can be seen in Table 3.10 that CTE of NSM FRP in transverse direction has consistent variation at elevated temperatures. For all the specimens investigated, transverse CTE attained relatively large values if larger temperature range was applied. Transverse CTE of Aslan GFRP attains $70 \times 10^{-6}/^{\circ}\text{C}$, whereas that of CFRP varies in a range of $(30 \sim 80) \times 10^{-6}/^{\circ}\text{C}$ for three different specimens. This level of thermal expansion in transverse direction does not cause significantly internal stress between FRP and concrete, since polymer matrix of FRP gets softened and melted beyond 300°C .

However, in longitudinal direction, thermal expansion might affect effective stress in NSM FRP when exposed to high temperatures. It can be seen that in Table 3.10 that in low temperature ranges (50°C or 100°C), data on longitudinal CTE has relatively larger variation for different specimens. Thus the data obtained from lower temperature range might not be reliable, and test results on larger temperature range are selected to evaluate the response of FRP under extreme conditions such as fire. Based on the test results in Table 3.10, longitudinal CTE of CFRP can be considered to be around $-5 \times 10^{-6}/^{\circ}\text{C}$ in $20 \sim 300^{\circ}\text{C}$ temperature range, and the corresponding values of GFRP is $3 \times 10^{-6}/^{\circ}\text{C}$.

3.4.4 Summary of thermal expansion tests

Coefficient of thermal expansion (CTE) of NSM FRP varies significantly depending on direction and composition. NSM GFRP has positive CTE (expansion) in both transverse and longitudinal directions. However, NSM CFRP expands in transverse direction at elevated temperatures, but shrinks in longitudinal direction. At relatively higher temperatures, GFRP and CFRP experience larger thermal expansion (or shrinking),

in both transverse and longitudinal directions. Based on measured data, CTE of GFRP and CFRP are recommended over a large temperature range (20-300°C) to evaluate the effect of thermal expansion to NSM FRP strengthened RC members.

Table 3.10 Transverse and longitudinal CTEs for various NSM FRP reinforcement

NSM FRP specimens		Longitudinal direction ($10^{-6}/^{\circ}\text{C}$)			Transverse direction ($10^{-6}/^{\circ}\text{C}$)		
		$\Delta T = 50^{\circ}\text{C}$	$\Delta T = 100^{\circ}\text{C}$	$\Delta T = 280^{\circ}\text{C}$	$\Delta T = 50^{\circ}\text{C}$	$\Delta T = 100^{\circ}\text{C}$	$\Delta T = 280^{\circ}\text{C}$
Aslan GFRP100	Test 1	3.5	1.3	2.6	28.2	52.9	74.8
	Test 2	5.3	1.7	2.8	28.0	49.9	68.5
	Average	4.4	1.5	2.7	28.1	51.4	71.7
Aslan CFRP200	Test 1	-10.3	-8.4	-13.0	10.6	12.1	18.3
	Test 2	-9.5	-9.7	-6.3	16.4	28.8	44.1
	Average	-9.9	-9.1	-9.6	13.5	20.5	31.2
Tyfo CFRP rod	Test 1	-1.7	-2.9	-4.3	37.6	56.9	67.4
	Test 2	-5.9	-7.5	-10.3	35.9	60.0	68.3
	Average	-3.8	-5.2	-7.3	36.7	58.5	67.8
Tyfo CFRP strip	Test 1	0.6	-0.8	-3.7	40.5	59.4	82.9
	Test 2	-0.4	-1.2	-1.2	26.4	51.1	73.0
	Average	0.1	-1.0	-2.5	33.4	55.2	77.9

3.5 Summary

Material property tests were performed to characterize various properties of NSM FRP at elevated temperatures. A large set of data was generated to gauge the effect of temperature on mechanical and deformation properties of NSM FRP, including tensile strength and modulus, bond strength and modulus, and thermal expansion. Data generated from these tests was utilized to develop empirical relations for mechanical and bond properties of NSM CFRP system as a function of temperature. The proposed empirical relations are capable of predicting mechanical and bond properties over a wide

temperature range. Thus, these relations can be used as input data in numerical models for evaluating fire response of NSM FRP strengthened members.

CHAPTER 4

FIRE RESISTANCE EXPERIMENTS

4.1 General

The literature review presented in Chapter 2 clearly shows that there is lack of experimental data on fire response of NSM FRP strengthened RC beams. No experiments have been carried out to evaluate fire resistance of NSM FRP strengthened RC beams without insulation. Critical factors influencing fire resistance such as load level, anchorage of FRP reinforcement, and fire induced axial force have not been quantified. To fill these knowledge gaps, fire resistance tests were undertaken on four NSM FRP strengthened T-beams. One beam was tested without any fire insulation, while the remaining three were protected with U-shaped insulation. These tests were aimed at generating reliable test data for validation of numerical models. Full details of the fire experiments, including beam fabrication, instrumentation, test procedure and measured response parameters, are presented in this chapter.

4.2 Preparation of Test Specimens

The test program consisted of design and fabrication of four NSM FRP strengthened RC T-beams and testing them under ASTM E119 standard fire conditions. The fabrication of test specimens comprised of three steps, namely, fabrication of RC T-beams, installation of NSM FRP, and installation of fire insulation.

4.2.1 Design and fabrication of RC T-beams

Four RC beams of T cross-section, representing beam-slab assemblies in buildings, were designed as per AIC 318 (2011) specifications. The dimensions of T-beams were selected to be close to typical building geometries. The flange of T-beams is of 432 mm in width and 127 mm in thickness, and the web is of 229 mm in width and 279 mm in depth. The beams have three 19 mm diameter rebars as flexural reinforcement and four 13 mm diameter rebars as compressive reinforcement. The stirrups used as shear reinforcement were of 6 mm diameter, and were spaced at 150 mm over the length of the beam and bent at the top flange at 135° into the concrete core. 13 mm diameter transverse rebars were placed at a spacing of 305 mm on the top of stirrups to prevent the failure of overhangs of beam flange (ACI 318 2011). The steel used for the main reinforcing bars and stirrups had specified yield strengths of 414 MPa and 280 MPa, respectively. The elevation and cross sectional details of T-beams are shown in Figure 4.1.

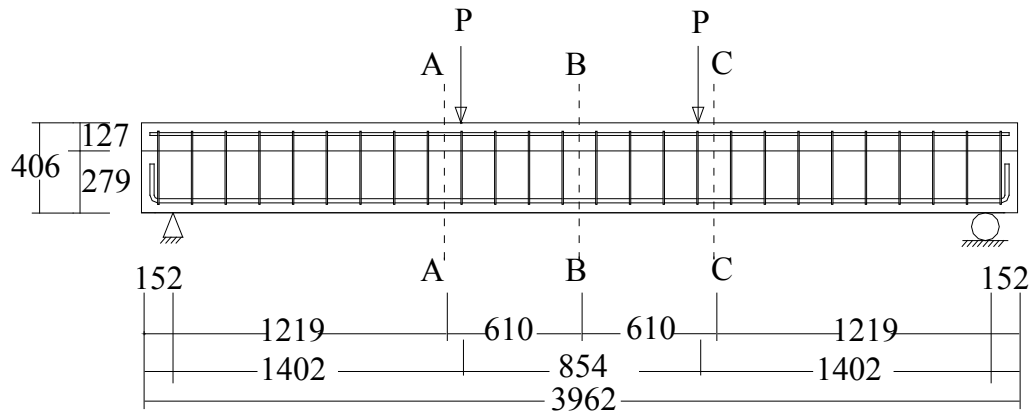
The above designed RC T-beams were fabricated at the Civil Infrastructure Laboratory in Michigan State University (MSU). Plywood forms were first designed and assembled to have the same internal dimensions as those of tested beams, as shown in Figure 4.2(a). Then the reinforcement cage (Figure 4.2(b)) was assembled and placed in a plywood form. All four beams were casted from one batch of concrete, which was supplied from a local batch mix plant to achieve good quality control. During pouring, concrete was vibrated and finished using concrete trowel to obtain smooth finishing surface. The concrete mix was designed to achieve a compressive strength of 41 MPa on 28th day. Type I Portland cement and carbonate based coarse aggregate were used in concrete batch mix. The measured compressive strength of concrete on 28th day was 48

MPa, and reached 50 MPa on 90th day. Batch proportions of concrete mix are given in Table 4.1.

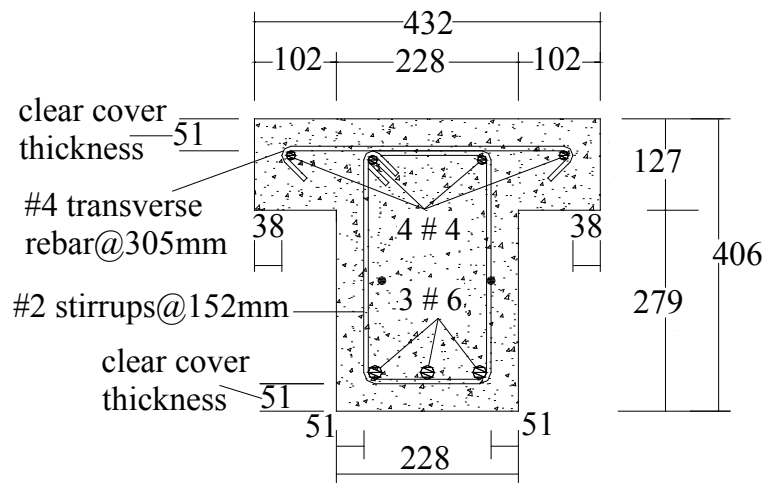
The casted beams were cured and sealed within the forms for three days, as shown in Figure 4.2(d). Thereafter, the beams were lifted out from the forms and stored in the laboratory, under a condition of 25°C temperature and 40% relative humidity.

Table 4.1 Batch proportion of concrete

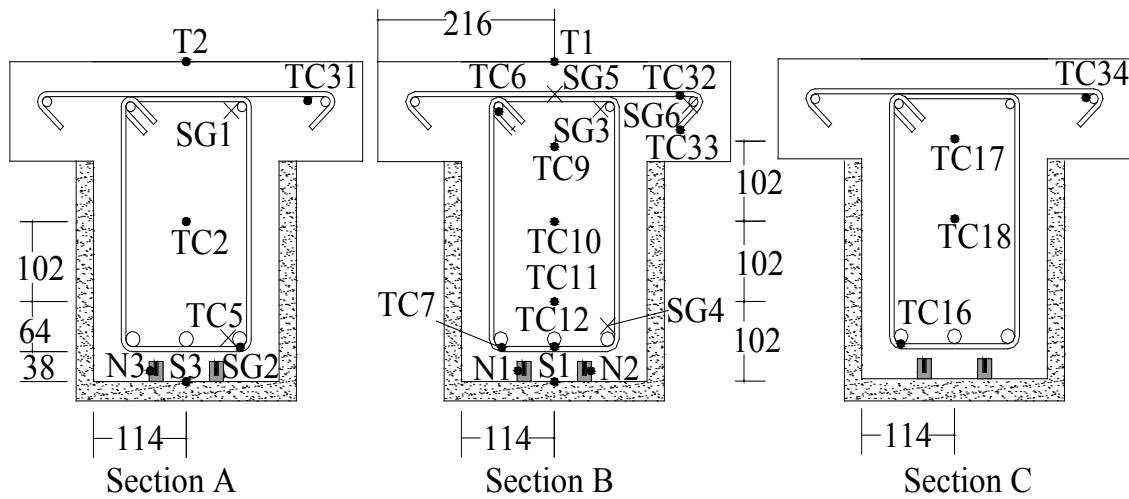
Cement, kg/m ³	309
Fine aggregate, kg/m ³	908
Course aggregate, kg/m ³	1015
Fly ash, kg/m ³	42
Slag, kg/m ³	59
Water, kg/m ³	160
Water cement ratio (w/c)	0.32
Air	6.5%
Moisture in fine aggregate	4%
Moisture in coarse aggregate	1%
Slump, mm	100
Unit weight of concrete, kg/m ³	85.3
Compressive strength f_c' (specified), MPa	41.4
Compressive strength f_c' (28 days), MPa	48
Compressive strength f_c' (90 days), MPa	50



(a) Elevation



(b) Cross section of RC beam



(c) Instrumentation

Figure 4.1 Elevation, cross-section, and instrumentation of FRP strengthened RC beams



(a) Preparation of wood forms



(b) Assembling reinforcement cage



(c) Casting of concrete



(d) Curing of fabricated T-beams

Figure 4.2 Steps in fabrication of RC beams

4.2.2 NSM FRP strengthening

4.2.2.1 Design of flexural strengthening

The flexural capacity of RC T-beams, which was computed to be 116 kN-m as per ACI 318 (2011), was enhanced by about 50% through strengthening using NSM CFRP strips. Typically in field application, the beam are strengthened to achieve 20-50% of additional capacity. To achieve this enhanced capacity, two Tyfo NSM CFRP strips were installed at the tension side of T-beam. Tyfo NSM CFRP strips are of high tensile strength and modulus, pull-formed, epoxy-carbon composite, and is usually applied together with Tyfo TC epoxy or thickened Tyfo S epoxy in NSM strengthening

applications. The cross-sectional area of NSM strip in use is $13.5 \text{ mm} \times 4.5 \text{ mm}$, and the length of strips is 3.18 m, which corresponds to outer dimension of the furnace. Thus the ends of NSM strips are thermally protected by the walls of furnace. This configuration was adopted to simulate the situation where anchorage zones of NSM strips are provided with thick insulation layers, or NSM strips are inserted into the partition walls (Firmo et al. 2010). Detailed properties of NSM CFRP strips are provided in Table 4.2.

The resulting moment capacity of NSM FRP strengthened RC beams was calculated to be 173 kN-m as per specifications prescribed in ACI 440.2 (2008). Detailed calculations of moment capacity of tested NSM FRP strengthened RC beams are presented in Appendix B.

Table 4.2 Properties of Tyfo NSM CFRP strips

Property	Typical test value	Design value
Dimension	$13.5\text{mm} \times 4.5\text{mm}$	$13.5\text{mm} \times 4.5\text{mm}$
Ultimate tensile strength in primary fiber direction	2.79 GPa	2.51 GPa
Elongation at rupture	1.8%	1.67%
Tensile modulus	155 GPa	139.6 GPa

4.2.2.2 Installation of NSM FRP strips

The installation of NSM FRP strips is as per the field application procedure provided by manufacture. Detailed installation procedures are listed as follows.

- Step 1: The beams were flipped upside down for ease of cutting the grooves. Two grooves, for placing two FRP strips, were cut on the soffit of each beam. The dimensions and spacing of grooves were as per ACI 440.2 specification (2008). The depth and width of the groove were 25 mm and 14 mm respectively, and the

clear edge distance between groove and beam edge was 70 mm, as shown in Figure 4.3.

- Step 2: After the cutting was finished, the grooves were cleaned using compressed air.
- Step 3: Before installation of NSM strips, Tyfo S epoxy, marketed by FYFE company, was prepared as filling adhesive. Generally Tyfo S epoxy is made by mixing two components (epoxy resin and hardener), as described in Section 3.2.1. When used in NSM applications, a third component, fumed silica (Cab-O-Sil), was added into epoxy, to thicken the adhesive as well as to provide stronger adhesion to CFRP strips during installation. These three components were mixed thoroughly using a mixer.
- Step 4: After the epoxy adhesive was uniformly blended, the adhesive was filled in the groove to its half depth.
- Step 5: An NSM CFRP strip was inserted into each NSM groove, and special attention was paid to position the CFRP strip at the center of the groove.
- Step 6: After positioning the CFRP strip, the entire groove was filled with epoxy adhesive.
- Step 7: After filling the grooves, the epoxy adhesive was cured for three weeks to achieve good bond between FRP strips and concrete substrate.

Various steps in the installation of NSM FRP is illustrated in Figure 4.4.

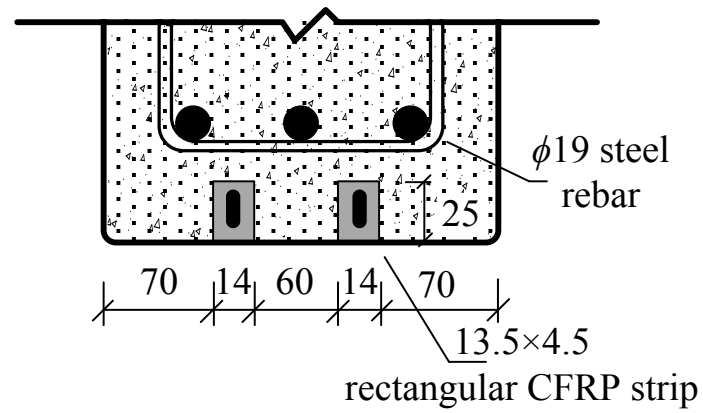


Figure 4.3 Location and dimensions of NSM grooves (Units: mm)

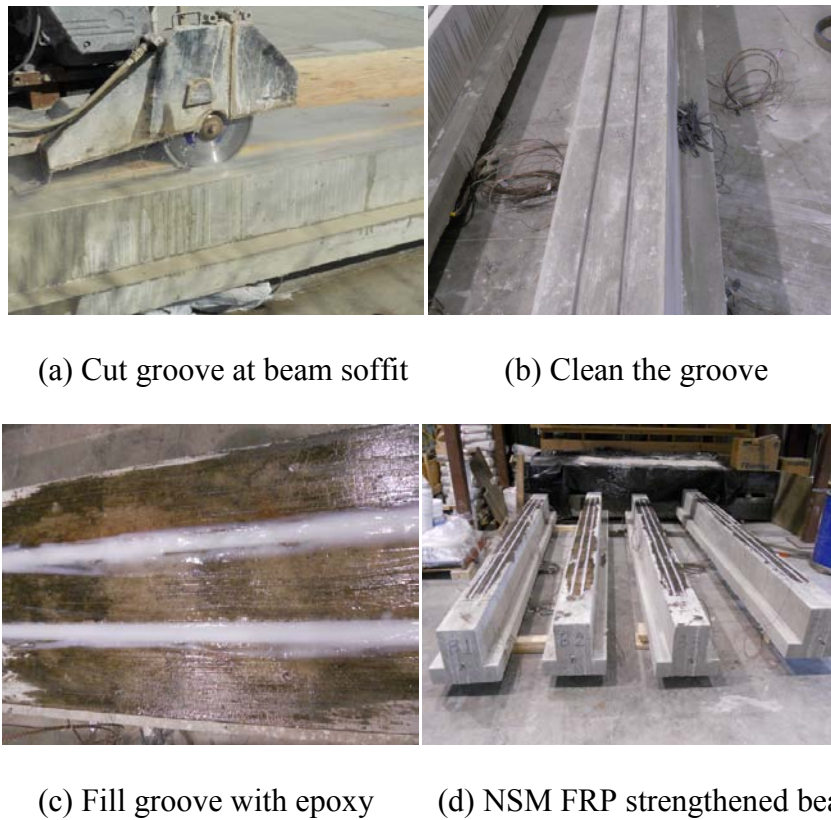


Figure 4.4 Installation of NSM FRP strengthening on RC T-beams

4.2.3 Fire insulation on T-beams

4.2.3.1 Fire insulation properties

To study the effect of fire insulation, three of the above strengthened beams were provided with Tyfo® CFP fire insulation. This Tyfo® CFP system, an improved version of previously developed Tyfo® AFP system (Fyfe 2013), comprises of three components; VG Primer, VG Dash Coat and WR-AFP. VG Primer is a special glue agent, which is applied on the concrete surface to provide better bond between concrete substrate and insulation material. VG Dash Coat is basically a sand coating, and it can roughen the concrete surface and thereby improve the adhesion of insulation material to the substrate. WR-AFP is the primary insulation material of Tyfo® CFP system. It possesses characteristics of lightweight, low thermal conductivity, and good crack resistance.

Tyfo® CFP system is non-combustible and non-flammable, and it provides up to 4 hours fire resistance rating. The density and bond strength of CFP insulation, as specified by manufacturer, is 458 kg/m^3 and 0.079 MPa, respectively (Fyfe 2013). The thermal conductivity and specific heat of CFP insulation are found to be 0.1936 W/m-K and 0.2698 MJ/kg-K, based on the tests conducted by Kodur and Shakya (2013).

4.2.3.2 Installation of fire insulation

The fire insulation on NSM FRP strengthened beams was applied by professional contractors from Fyfe Company. The installation procedure is illustrated in Figure 4.5. The first step of installation was spraying a layer of VG primer on cleaned concrete surface (see Figure 4.5(a)). The VG primer layer is to be applied uniformly to cover the entire beam substrate, since any defects could result in debonding of insulation. Then a thin layer of dash coat was sprayed on VG primer layer (see Figure 4.5(b)); this is mainly used to roughen concrete surface to ensure better adherence of insulation layer. After

applying the above two layers, the beams were cured for 2-3 hours so as to generate good bond between insulation and concrete substrate.

Thereafter, Tyfo® WR-AFP, which is usually in powdered form, was mixed with appropriate amount of clear water and spray-applied on beams using a hopper gun, as shown in Figure 4.5(c). The mixed material was applied in lifts of approximately 8-10 mm thickness to accelerate the drying procedure before next lift was sprayed. During application, the thickness of insulation was measured at several places along the beam length to maintain uniform thickness throughout the depth and length of beam web. The finished insulation system was 25 mm thick at the bottom surface of beam web and extended to 200 mm on two sides of web (see Figure 4.6). The insulation was cured for 21 days to ensure that full bond strength of insulation material is developed. The complete insulated beams are shown in Figure 4.5d.



(a) Application of a VG primer layer



(b) Spray a dash coat layer



(c) Spray a WR-AFP layer



(d) Insulated beams for curing

Figure 4.5 Steps in application of fire insulation on NSM FRP strengthened RC beams

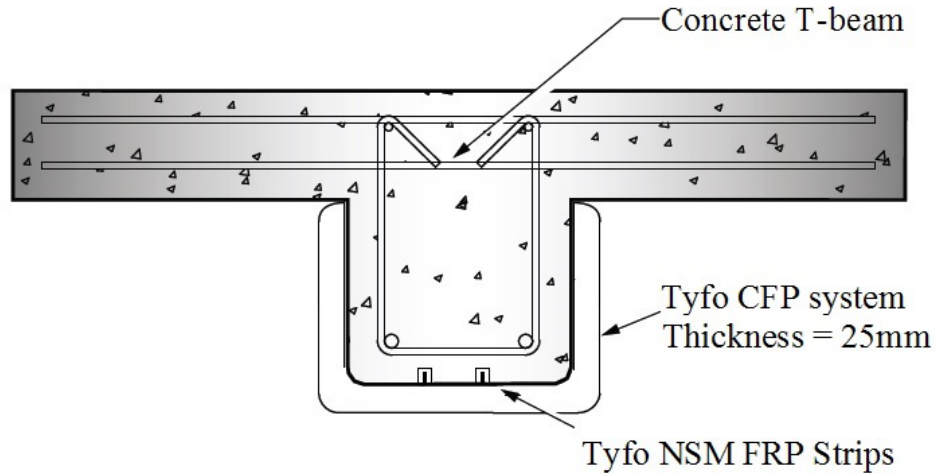


Figure 4.6 Layout of fire insulation scheme on NSM FRP strengthened RC beams

4.2.4 Instrumentation

The instrumentation mounted in strengthened T-beams included thermocouples, strain gauges and displacement transducers. To monitor temperature progression within beams, Type-K Chromel-alumel thermocouples were installed at three different sections in each beam. During the fabrication of RC T-beams, each beam was instrumented with 23 thermocouples so that temperatures at various locations of concrete and steel rebar can be recorded during the tests. During the installation of NSM FRP, thermocouples were bonded at mid-span, quarter span, as well as two ends of FRP strip, to monitor the variation of temperature in FRP strips and anchorage zones. Some other critical locations, such as unexposed surface (beam top), beam-insulation interface, were also installed with thermocouples, as shown in Figure 4.1(c).

Normal temperature strain gauges were mounted to compression and tension rebars. These strain gauges were bonded to flat finished surface of steel rebar, and protected with a small piece of duct tape to minimize damage during the casting of

concrete. The location and numbering of thermocouples and strain gauges in the cross-section are shown in Figure 4.2c.

In addition, three Linear Variable Differential Transducers (LVDTs) were installed at unexposed surface (top) along the centerline of each beam, one at mid-span and two at loading cells to measure the deflections of beam during fire tests. For the beam with axial restraint, one additional LVDT was applied at one support of the beam to record variation of axial displacement in the beam.

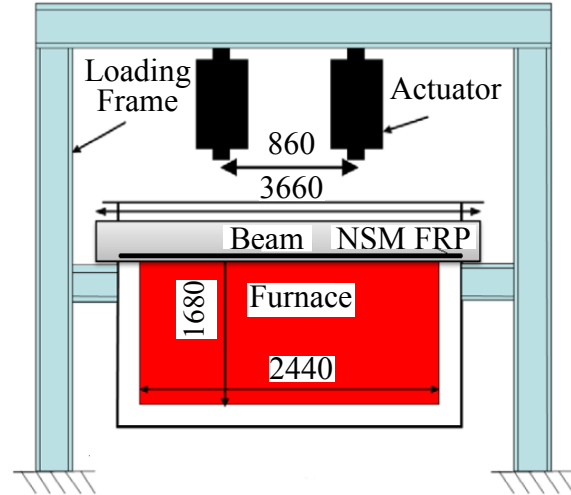
4.3 Test Apparatus

The fire resistance tests on NSM FRP strengthened beams were carried out using the structural fire testing furnace in the Civil Infrastructure Laboratory at Michigan State University. The test furnace, shown in Figure 4.7, has the capacity to supply both heat and loading that are representative to those in a typical building exposed to fire. The furnace consists of a steel frame supported by four steel columns, with a fire chamber that is 2.44 m wide, 3.05 m long, and 1.78 m high. Six natural gas burners are located within the furnace and provide thermal energy, and the maximum heat (power) can reach 2.5 MW. Six Type-K thermocouple probes placed as per ASTM E119 (2008), are distributed throughout the test chamber, and they are used to monitor the furnace temperature during a fire test. During the fire test, these furnace temperatures are used to manually adjust fuel supply, and maintain a temperature time curve consistent with a pre-determined standard or design fire scenario. In this way, the furnace temperature can be maintained along a desired curve. Two small view ports on either side of the furnace wall are

provided for visual monitoring of the fire-exposed specimens during a test. The furnace facilitates two beams at a time, and different load levels can be applied on each beam.



(a) Furnace and loading frame



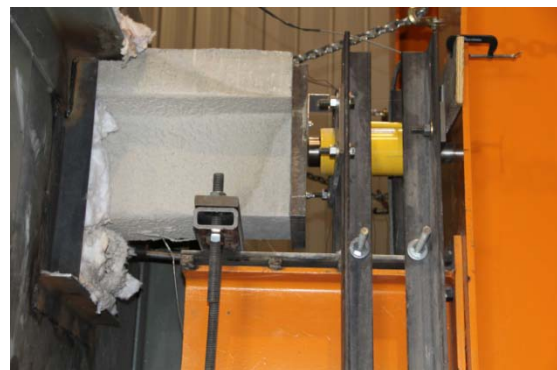
(b) Schematic for front view of furnace

Figure 4.7 Structural fire test furnace at MSU Civil and Infrastructure Laboratory

The axial restraint was applied on one beam during the fire test (Beam III). The devices used for simulating axial restraint are as shown in Figure 4.8. One end of the beam was loaded through a hydraulic jack, ENERPAC RC-506, and the other end of beam was connected to steel frame through a short steel beam. The loading capacity of hydraulic jack is 498 kN, and the maximum stroke is 159 mm.



(a) Axial restraint at one beam end



(b) Axial restraint at the other beam end

Figure 4.8 Installation of axial restraint on NSM FRP strengthened RC beam (Beam III)

Data from the test which included temperatures, displacement, strains, and forces, was collected through “Darwin Data DA100/DP120-13” data acquisition system. This system is capable of recording 70 thermal couple channels, 10 strain gauge channels, and 10 LVDT channels. All these channels were connected to data acquisition system and the measurements in the tests were recorded in “.CSV” file using “DAQ32” computer program.

4.4 Test Conditions and Procedure

During each fire experiment, two NSM FRP strengthened RC T-beams were tested simultaneously under loading and fire conditions, and thus two fire experiments were carried out (on four beams). In both fire tests, the beams were simply supported at ends with an unsupported length of 3.66 m, of which 2.44 m was exposed to fire in the furnace. ASTM E119 standard fire was applied in both fire tests. The experimental program and variables studied in fire tests are shown in Table 4.3.

In the first fire test, one uninsulated RC beam (Beam I) and one insulated RC beam (Beam II) were tested. This is to gauge the fire resistance of NSM FRP strengthened RC beam without any protection, as well as to investigate the effect of insulation to fire response of strengthened RC beams. In the second fire test, the effect of boundary conditions was evaluated through adding axial restraint to one of the two tested beams. Also, different loads were applied in two fire tests and thus the effect of load level was studied.

All four beams were subjected to two-point loading in fire tests, and each point load was 1.4m away from the end support, as shown in Figure 4.1(a). Concentrated loads

of 62 kN and 80 kN (loading at one point) were applied in the first and second fire test respectively, and they represent 50% and 65% of nominal capacity of the strengthened beam at room temperature. This nominal capacity was determined as per ACI 440.2R (2008) that requires the effective strain in NSM FRP should be limited to certain level to prevent the debonding of FRP from concrete substrate. Thus, the moment capacity was computed with this limiting strain to obtain the superimposed loading. Details of calculations are provided in Appendix B.

In fire tests, the loading was applied approximately 30 minutes before the start of the test until steady condition (no increase in deflection with time) was reached. This was selected as the initial condition for the deflection of the beam.

Table 4.3 Variables studied in fire tests on NSM FRP strengthened T-beams

Fire tests	Beam specimens	Insulation	Load (ratio)	Boundary conditions
1 st test	Beam I	None	62 kN (50%)	Simply supported
	Beam II	U-shaped	62 kN (50%)	Simply supported
2 nd test	Beam III	U-shaped	80 kN (65%)	Simply supported with axial restraint
	Beam IV	U-shaped	80 kN (65%)	Simply supported

4.5 Material Tests

Material tests on constituent materials of NSM strengthened T-beams, including concrete, steel rebar, FRP strip, were carried out to obtain respective strength properties. To determine compressive strength of concrete, concrete cylinders prepared from the same batch mix, as that used for fabricating concrete beams, were tested at 7, 28 days, 90 days, and on the day of fire testing. Average compressive strength of concrete is tabulated in Table 4.4. The 28-day and 90-day compressive strength of the concrete was 48 and 50 MPa, respectively, which is higher than the design compressive strength of 41.4 MPa.

Table 4.4 Compressive strength of concrete

Test date	Design strength	7-day	28-day	90-day	Test day
Concrete compressive strength (MPa)	41.4	35.7	48.0	49.7	53.1

The yield strength and ultimate strength of steel rebars were obtained through tensile strength tests using 810 universal material testing system (MTS). Two tensile tests were undertaken on rebar samples of 19 mm diameter. The average yield strength, ultimate strength and ultimate strain were found to be 455 MPa, 674 MPa and 0.18, respectively. The stress-strain curves for the tested rebars are shown in Figure 4.9.

The mechanical properties of NSM CFRP strip and bond properties of NSM adhesives (Tyfo S epoxy), were also evaluated through tests. Details on test procedure and results of these tests are presented in Chapter 3.

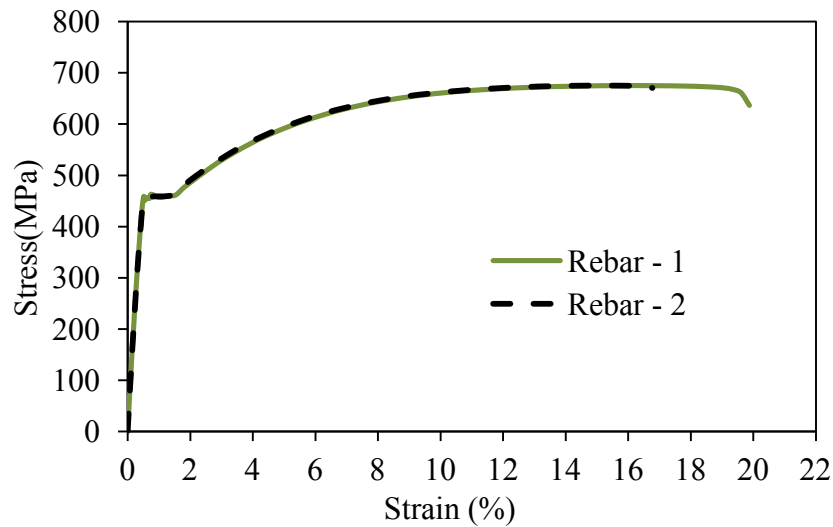


Figure 4.9 Stress-strain relations of steel rebars used for flexural reinforcement

4.6 Test Results and Discussion

A large set of test data was collected in fire resistance tests, including temperatures at various locations, strains in compression and tension rebars, mid-span deflections of beams, and axial restraint forces. This data was utilized to evaluate the comprehensive fire behavior of NSM FRP strengthened RC beams. Also, the response parameters of NSM FRP strengthened RC beams were compared with published test data on conventional RC beam and externally bonded FRP strengthened RC beam. Detailed information on observations taken during fire tests, thermal response, structural response, and residual strength capacity of these beams are discussed in the following sections.

4.6.1 Test observations

During fire tests, visual observations were recorded from two viewing windows on each opposite side of the furnace walls. Important events during the tests, including insulation cracking, epoxy burning, beam cracking, were recorded through photographs and videos. Tables 4.5 and 4.6 provide a summary of observations at critical moments in two fire tests.

In the first fire test, NSM epoxy on uninsulated beam (Beam I) started burning after only 10 minutes into fire exposure. This is mainly due to highly combustible nature of epoxy, and results of direct exposure to fire. Burning of epoxy on Beam I lasted for about 50 minutes. The polymer matrix of CFRP strips mostly burned out, and some carbon fibers were exposed at the beam soffit, as shown in Table 4.5 (80 minutes). However, since the anchorage zones of NSM FRP strips were protected by furnace wall, most carbon fibers stayed inside of NSM grooves, and no significant detachment of these

fibers was seen during the test. At later stages of fire, some cracks developed on Beam I, however, the beam did not fail for 210 minutes of fire exposure.

Compared to Beam I, epoxy in Beam II did not burn in the early stage of fire exposure, as U-shaped fire insulation well protected this beam (from bottom and sides). There was a little burning of epoxy at beam soffit starting at 60 minutes, and this is due to onset of cracking in fire insulation. Throughout the fire test, epoxy burning in Beam II did not cease and it turned more severe at later stage of the test. However, Beam II did not fail during 210 minutes of fire exposure, but this insulated beam exhibited much better performance than that of uninsulated beam (Beam I).

In the second set of fire test, Beams III and IV were subjected to higher loading (65% of room temperature moment capacity) than that in Beams I and II. Thus, cracking of insulation occurred earlier in these two beams as compared to Beam II, and thus the burning of epoxy was more severe. At later stage of the test (160 minutes), one piece of fire insulation fell off from the soffit of Beam IV (without axial restraint), and epoxy at that unprotected area quickly burned out. In contrary, the insulation on Beam III remained attached to soffit and sides of beam during the entire fire exposure, and this is mainly attributed to relative smaller deflection resulting from axial restraint at the supports. At last, Beams III and IV did not fail during 210 minutes of fire exposure. The critical observations at various timelines, together with photos, are presented in Tables 4.5 and 4.6.

Table 4.5 Visual observation for Beams I and II in the first fire resistance test

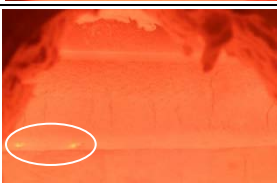
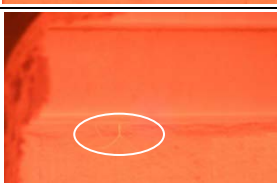
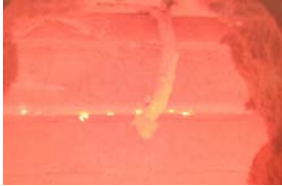
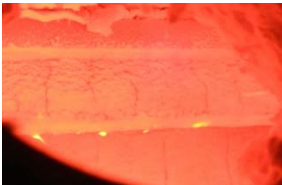
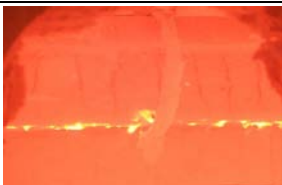
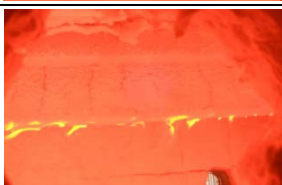
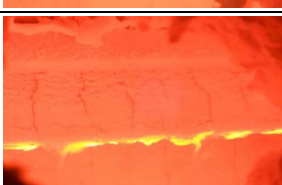
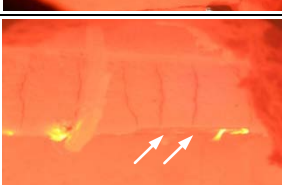
Time (mins)	Observations	State of the specimens
0	Beams were loaded to 50% of their room temperature capacity. ASTM E119 standard fire started.	
10	NSM epoxy on Beam I started burning.	
25	The entire beam soffit of Beam I was engulfed in fire.	
35	Vertical cracks occurred in the insulation of Beam II.	
60	Cracking of insulation appeared at bottom of Beam II, and epoxy on Beam II started burning.	
80	Burning of epoxy in Beam I stopped. Some carbon fibers exposed at beam soffit.	
100	Burning of epoxy was observed at multiple spots at the bottom of Beam II.	
190	Visible cracks appeared on Beam I	

Table 4.6 Visual observation for Beams III and IV in the second fire resistance test

Time (mins)	Observations	State of the specimens
0	Beams were loaded to 65% of their room temperature capacity. ASTM E119 standard fire started.	
30	NSM epoxy on Beam IV started burning.	
55	Cracking of insulation appeared on Beam III, and epoxy at beam soffit started burning.	
65	More cracks occurred in the insulation of Beam IV, and burning of epoxy got severe.	
80	More cracks occurred in the insulation of Beam III, and burning of epoxy occurred at multiple spots.	
160	One piece of insulation at the bottom of Beam IV fell off, and the epoxy around the protected area burned severely.	
190	On Beam III, burning of epoxy got severe.	
195	NSM epoxy at the place where insulation fell off on Beam IV completely burned out.	

4.6.2 Thermal response

During the fire test, temperatures at various locations within beam cross section were recorded to evaluate thermal response of NSM FRP strengthened RC beams. This section presents details on temperature progression at critical points including that on NSM FRP, insulation/concrete interface, steel rebars, and various depths of concrete.

4.6.2.1 Furnace temperatures

Figure 4.10 presents the temperature-time curve of ASTM E119 standard fire and measured average furnace temperatures in two fire tests. The beams were exposed to ASTM E119 standard fire for 210 minutes. Overall, it can be seen that furnace temperatures reasonably match the required standard fire temperature, and the discrepancy between average furnace temperature and ASTM E119 fire is within 5% range throughout fire duration. This ensures that two sets of beams were tested under similar fire exposure conditions, and test results of these beams are comparable.

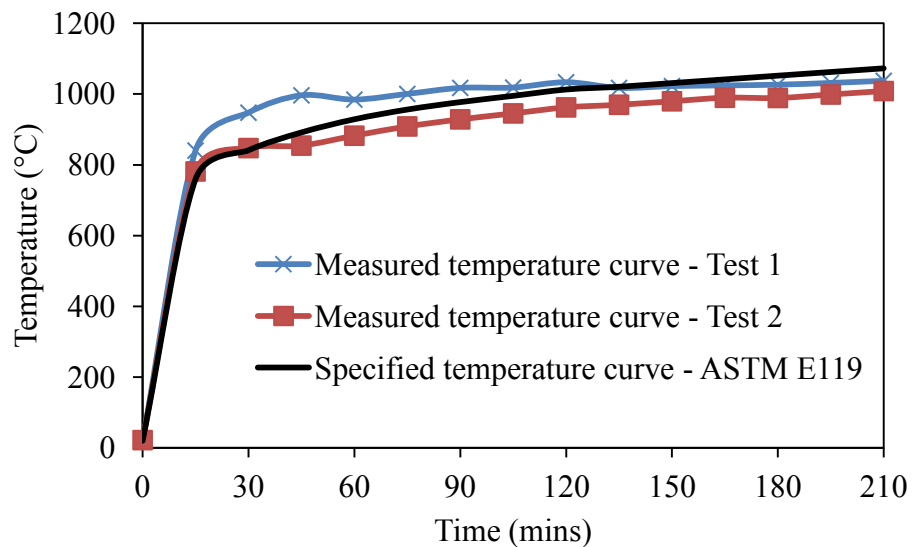


Figure 4.10 Measured and specified time-temperature curve during fire tests

4.6.2.2 NSM FRP temperatures

Temperature in NSM FRP strips is an important indicator of the condition of FRP under fire exposure. In this test program, thermocouples were bound to CFRP strips at various locations and inserted into NSM grooves. Figure 4.11 shows temperature rise in NSM FRP strip for each tested beam. In Beam I, it can be seen that temperature in FRP jumped to 600°C at about 20 minutes into fire, and then reached 800°C at about 40 minutes. This quick rise in temperature is mainly caused by severe burning of epoxy in NSM grooves, as shown in Table 4.5. The burning of epoxy spread along the entire length of the beam, and thus FRP temperatures at mid-span and quarter span were equally high during the test. Thus in later stage of fire exposure, CFRP strips turned into carbon fibers to support the beam. In the anchorage zone, since the epoxy and FRP strips were not exposed to fire directly, temperature in FRP strips at both ends remained lower than 100°C. This cool anchorage ensures NSM FRP continued to contribute to load bearing capacity of Beam I, even though temperature in some places of FRP strips went beyond 800°C.

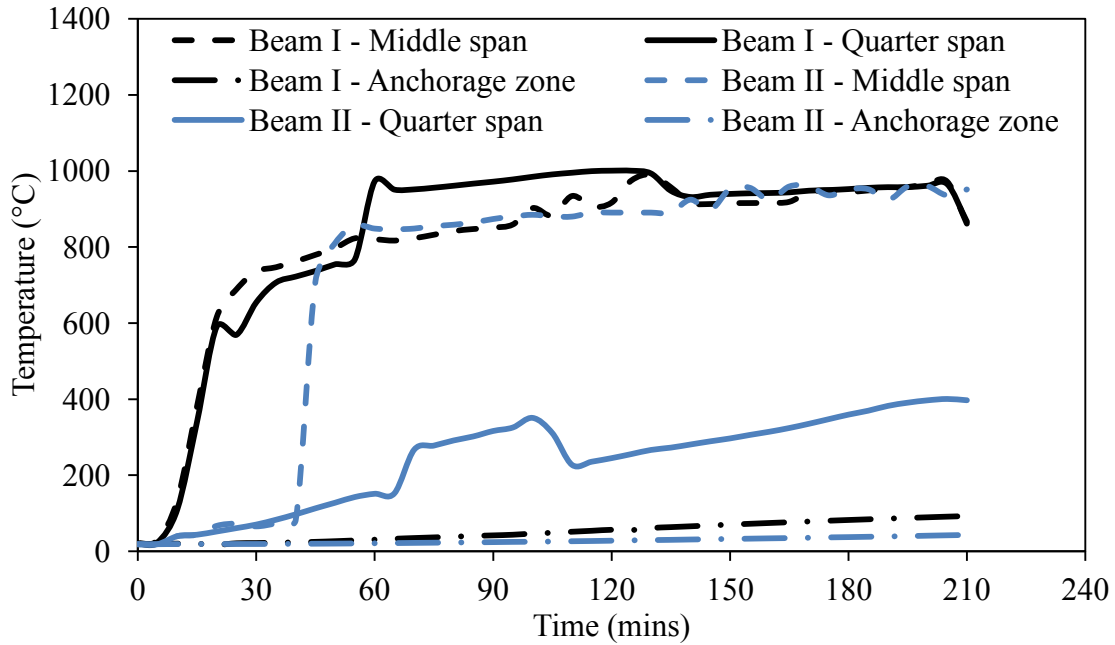
Owing to the protection from U-shaped fire insulation, temperatures rise in FRP in Beam II was in a much slower rate than that in Beam I. As shown in Figure 4.11(a), the temperature at quarter span of FRP increased at a slow pace for the entire fire duration. After 210 minutes of fire exposure, FRP temperatures measured at quarter span only reached 400°C. However, FRP temperatures at mid-span were much higher in fire test. After 40 minutes, due to cracking occurred in the insulation, temperature at mid-span of FRP jumped to 800°C within 10 minutes, and remained in 800-900°C range in the rest fire duration. However, this localized high temperatures did not lead to debonding of

NSM FRP strips in the test, since temperatures in other parts of NSM FRP strips remained low.

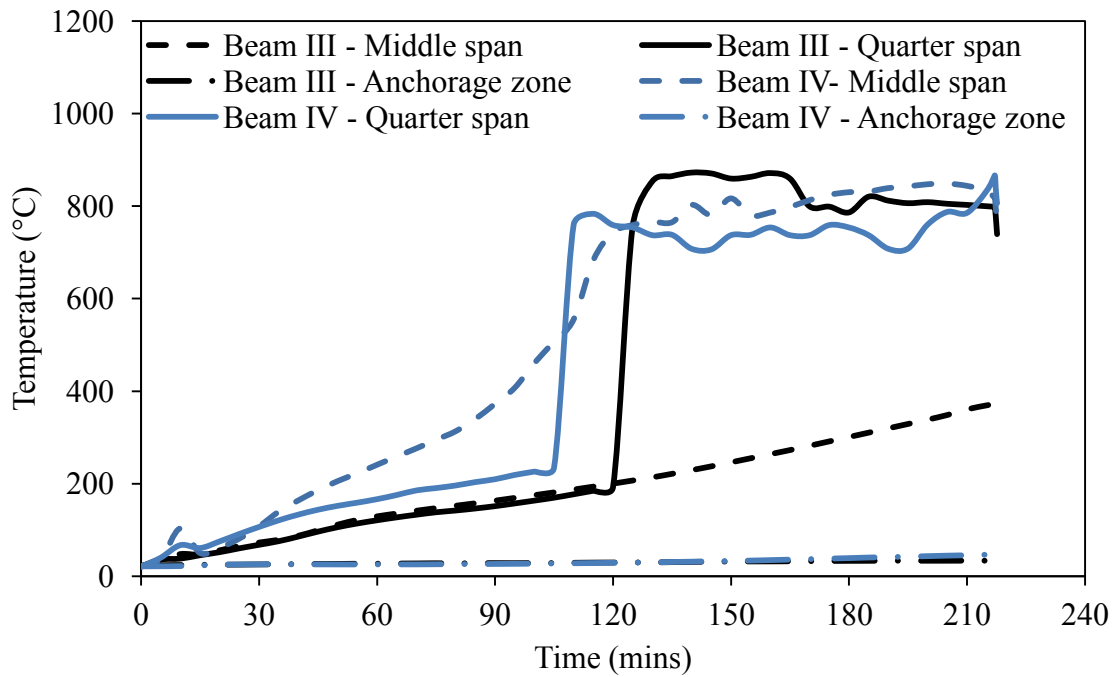
The temperature rise in NSM FRP in Beams III and IV was similar to that in Beam II, since the same U-shaped fire insulation was applied on these three beams. As shown in Figure 4.11(b), FRP temperatures in these two beams remained lower than 300°C until 100 minutes, both at middle and quarter span. At 100 minutes, NSM FRP in both beams attained high temperatures (800°C), and this is directly related to crack formation in the insulation. Depending on the size of crack and location of thermal couples, NSM FRP temperatures can be significantly different. Temperatures in NSM FRP in Beam III were relatively lower than those in Beam IV. This lower temperature is mainly attributed to the fact that presence of axial restraint on Beam III minimized the cracks generated in the insulation, and thus heat penetration through insulation was reduced.

Temperatures at insulation/concrete interface were also recorded at various locations along the length of insulated beams (Beams II-IV), as shown in Figure 4.12. It can be seen that temperatures at insulation/concrete interface are similar for three insulated beams: temperature rise is relatively faster in first 40 minutes of fire exposure, and then gradually increases until the end of fire exposure. Compared with NSM FRP temperature curves in Figure 4.11, temperatures at insulation/concrete interface (outside of groove) were slightly higher than those in NSM FRP (inside of groove). This indicates NSM epoxy and concrete cover have certain thermal protection effect to NSM FRP. However, if epoxy in NSM groove started burning, the temperatures at insulation/concrete interface also jumped to high level (800-900°C). Thus temperatures at

NSM FRP and at insulation/concrete interface were essentially the same as fire temperatures.



(a) Beam I and Beam II



(b) Beam III and Beam IV

Figure 4.11 Variation of NSM FRP temperatures with fire exposure time in Beams I-IV

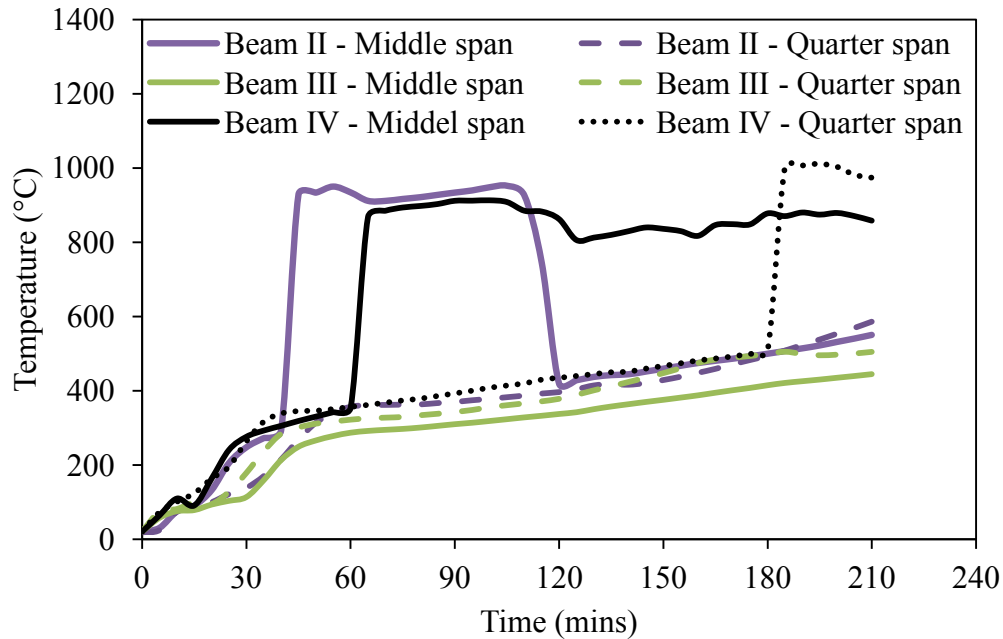


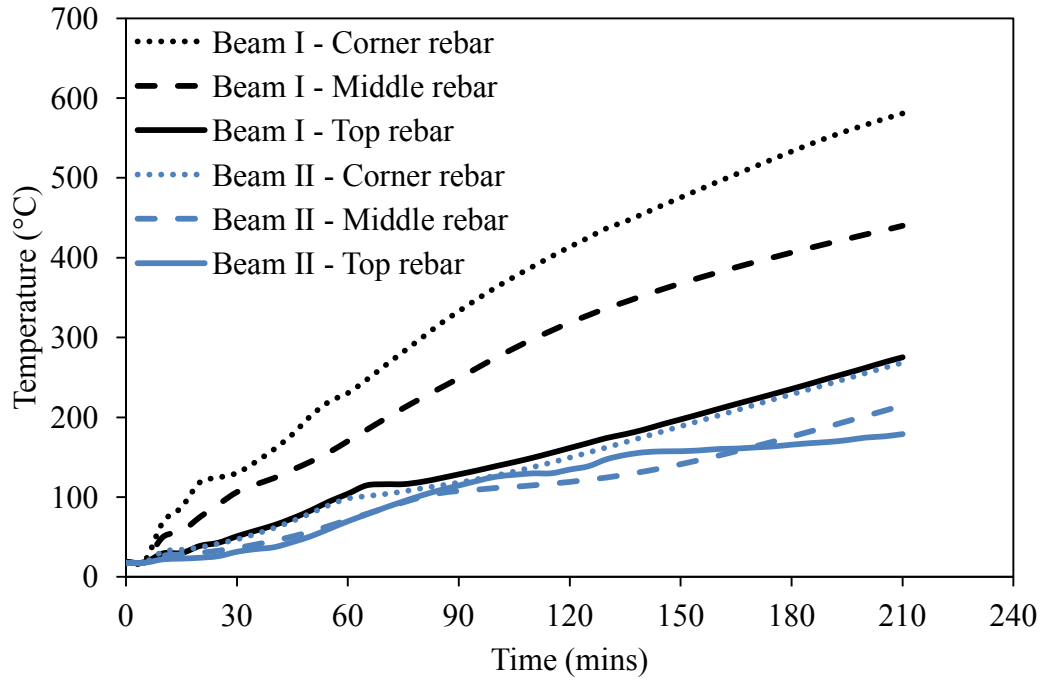
Figure 4.12 Variation of temperatures at insulation/concrete interface with fire exposure time in Beams II-IV

4.6.2.3 Steel rebar temperatures

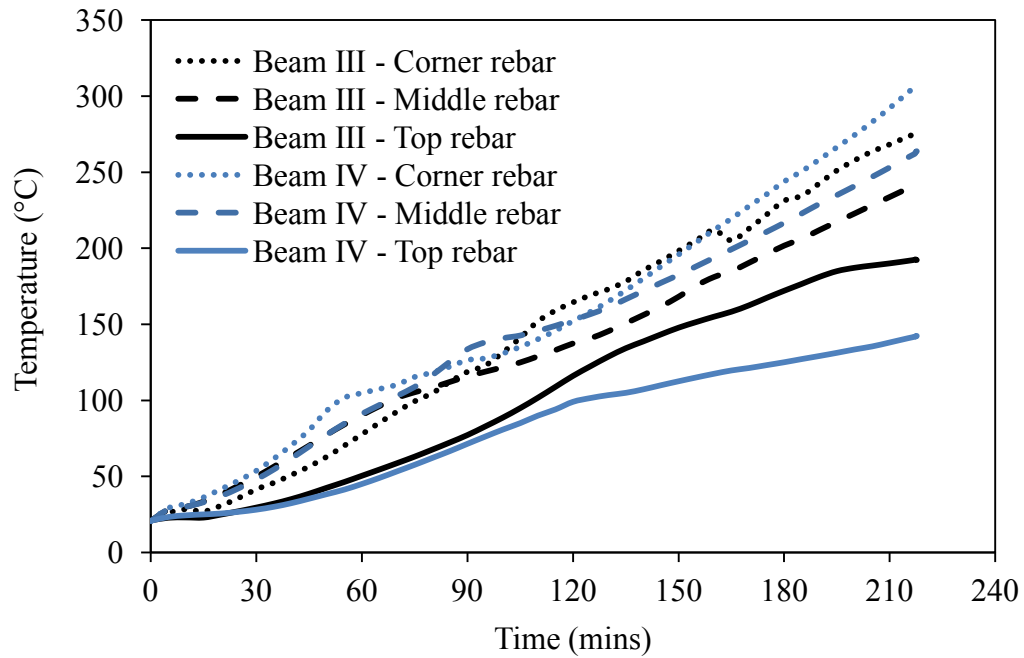
Since FRP reinforcement usually experiences faster degradation on strength and stiffness during fire exposure, strength retention in steel rebars plays a critical role on fire resistance of FRP strengthened RC beams. Therefore, temperatures in rebars were monitored throughout the fire tests. Figure 4.13 shows variation of rebar temperature as a function of fire exposure time for four tested beams. It can be seen that rebar temperatures in Beam I increased at much higher rate than those in other beams. At 210 minutes, corner rebar in Beam I reached about 600°C, and this already exceeds the temperature limit (593°C) specified in ASTM E119 (2012). However, temperatures in middle rebar attained 400°C, indicating it still possessed most of the original strength. That's one reason why Beam I did not fail in the fire test.

For Beam II, rebar temperatures were much lower than those in Beam II at any given fire exposure time, mainly due to protection of fire insulation. It can be seen that temperatures in corner and middle rebars remained below 300°C during entire fire exposure, so there was not much strength degradation in steel rebars of Beam II. This indicated that U-shaped fire insulation can effectively reduce heat progression within the beam.

In the case of Beams III and IV, temperature in steel rebars rose at slightly higher rate than that in Beam II, as shown in Figure 4.13(b). This can be attributed to cracking developing and widening on insulation layer which resulted from higher loading (65% of load ratio). At later stage of fire exposure, the cracks in the insulation got enlarged, and one piece of insulation in Beam IV even fell off during the test, and this introduced more heat transfer into the beam. However, both corner and middle rebar temperatures still remained below 350°C throughout the fire exposure, which indicated only little strength loss occurred in steel rebars.



(a) Beams I and II



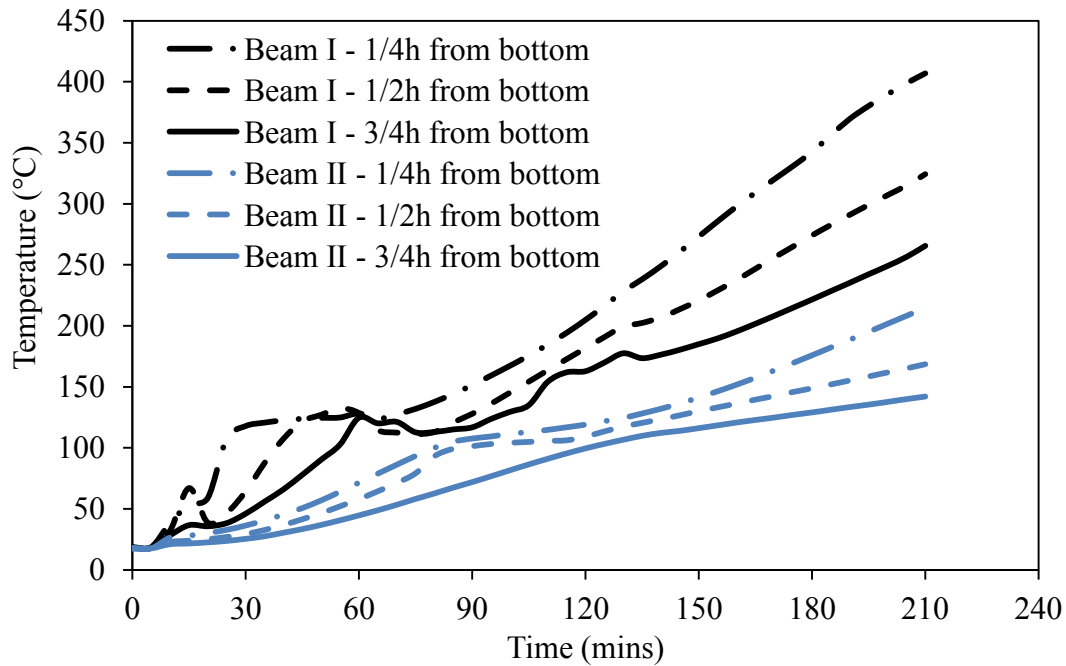
(b) Beams III and IV

Figure 4.13 Variation of steel rebar temperatures with fire exposure time in Beams I-IV

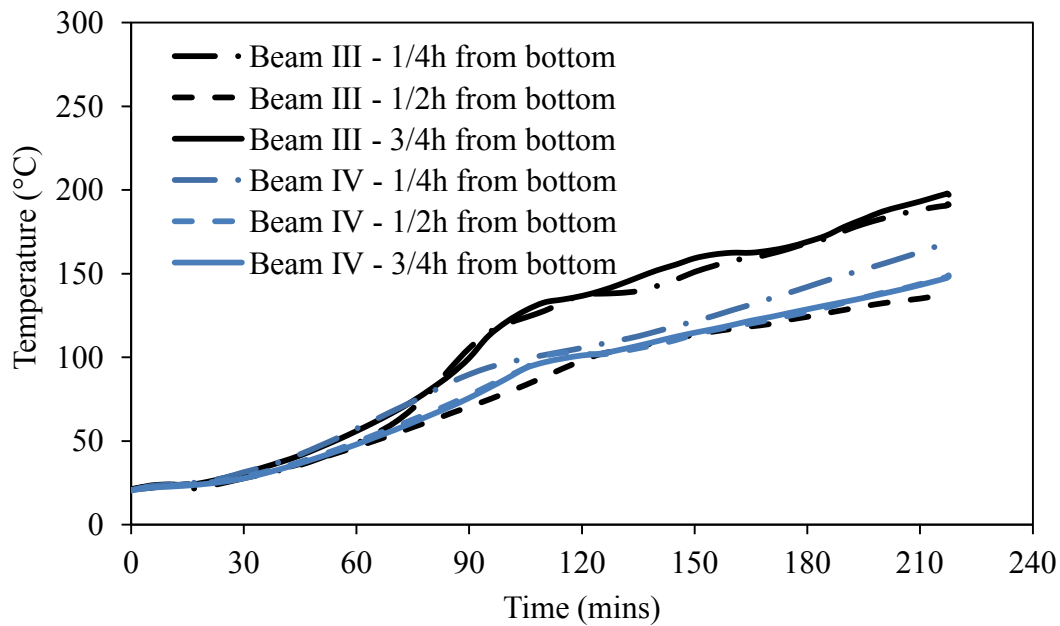
4.6.2.4 Concrete temperatures

The variation of concrete temperatures at various depths is plotted in Figure 4.14, as a function of fire exposure time. The locations monitored during fire exposure include concrete at quarter depth, mid-depth, and three quarters depth from beam soffit. As expected, concrete at locations closer to fire exposure attained relatively higher temperature. In Beam I, temperatures at various depths of concrete increased to 100°C at about 30-40 minutes, and then sustained a plateau at around 100°C for more than 30 minutes. This is due to the fact that moisture in concrete absorbs significant heat during evaporation process. After this moisture got evaporated, temperature in concrete continued to increase. At last stage of fire exposure, concrete temperature at quarter depth from beam soffit reached 430°C. However, temperatures in the upper half concrete, which is the primary compression zone of beam, still remained below 300°C.

Temperature rise in concrete in insulated beams (Beams II-IV) was very slow throughout fire exposure duration. It can be seen that in Figure 4.14(b) that there is no significant temperature gradient developed over the depth of concrete, and the highest temperature attained in concrete is only 200°C. This is mainly attributed to the fact that U-shaped insulation covers the entire web of beam, and thus thermal propagation within concrete section is considerably reduced. Based on temperature-strength relationship specified in ASCE (Lie 1992) and Eurocode (2004), there is no strength and stiffness loss in concrete until 400°C. Thus, in all four tested beams, concrete retained most of the nominal strength and stiffness throughout fire exposure duration.



(a) Beams I and II



(b) Beams III and IV

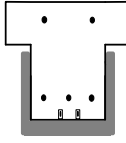
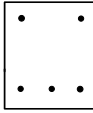
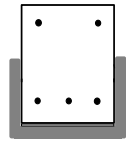
Figure 4.14 Variation of concrete temperatures with fire exposure time at various locations in Beams I-IV

4.6.3 Structural response

4.6.3.1 Deflections

Structural response of four NSM FRP strengthened RC beams is compared in Figure 4.15, by plotting the variation of mid-span deflection as a function of fire exposure time. Previously test data on an unstrengthened RC beam tested by Dwaikat and Kodur (2009) and an externally FRP strengthened RC beam tested by Ahmed and Kodur (2011) are also plotted in the figure to compare relative fire response of RC beams with different FRP strengthening. These two beams are of 254×406 mm rectangular sections, which are slightly wider than tested T-beams. However, all beams are in the same depth and comprise of same flexural reinforcement, and further all beams were tested exposed to ASTM E119 fire. Thus, the behaviors of these beams are comparable. The configurations of unstrengthened RC beam, external FRP strengthened RC beam, as well as NSM FRP strengthened T-beams, are shown in Table 4.7.

Table 4.7 Configuration and test conditions of RC beams with various FRP strengthening

Beams	Beam configuration			Load ratio	Fire insulation
	Steel rebars	Cross section (mm)	Figure		
Beam I	3 ϕ 19 mm in tension	Flange 432×127 Web 228×279		50%	None
Beam II				50%	25mm U-shape
Beam III	4 ϕ 13 mm in compression			65%	25mm U-shape
Beam IV				65%	25mm U-shape
Unstrengthened RC beam	3 ϕ 19 mm in tension 2 ϕ 13 mm in compression	254×406		54%	None
External FRP strengthened RC beam	3 ϕ 19 mm in tension 2 ϕ 13 mm in compression	254×406		50%	25mm U-shape

The variation of mid-span deflection as a function of fire exposure time for four tested T-beams is plotted in Figure 4.15. It can be seen that the uninsulated Beam I experienced much faster deflection than those of insulated beams (Beams II to IV). Due to direct exposure to fire, NSM epoxy in Beam I started burning at 10 minutes into fire. Thus the bond between FRP and concrete member in Beam I was severely affected at the early stage of the fire exposure, and FRP strength and stiffness also decreased significantly. These are two main factors resulting in much larger deflections in Beam I. Thereafter, the deflection keeps increased during the fire exposure, due to temperature induced strength degradation in steel rebar and concrete. To prevent sudden failure of beam, the loading on Beam I was released a little in the final stage of the test, and thus the deflection in Beam I almost stopped increasing. Finally Beam I did not fail for 210 minutes of fire exposure.

Due to the protection of fire insulation, Beams II-IV underwent lower deflections as compared to Beam I. Beams II retained a small amount of deflection in the entire fire exposure duration, and this mainly results from low temperatures in steel rebar and NSM FRP (refer to Section 4.6.2). For Beams III and IV, the mid-span deflections also remained in a low level in the first 90 minutes. However, after 90 minutes, these two beams experienced accelerating deflections, and this is mainly due to bond degradation of NSM FRP resulting from burning of epoxy. Since the loading on Beams III and IV is relatively large (65% of room temperature capacity), more cracks developed in the insulation, and thus more epoxy burning is induced on these two beams. However, Beam III yielded smaller deflection as compared to Beam IV. This is mainly attributed to the counteracting moment developed through axial restraint force, which reduced the

moment applied by external loading. This effect is similar to that of prestressed strands to a concrete beam.

The comparative deflection response of RC beams with different FRP strengthening is also plotted in Figure 4.15. It can be seen that deflection response of NSM FRP strengthened RC beam without insulation (Beam I) is similar to that of unstrengthened RC beam tested by Dwaikat and Kodur (2009). However, NSM FRP strengthened RC beam experienced smaller deflection than that of unstrengthened RC beam throughout fire duration. This can be attributed to the “cable action” that was developed through remaining NSM CFRP strips. It is established that carbon fibers possess good temperature resistance and can retain most of their strength even at 1000°C (Davies et al. 2004, Sauder et al. 2004). Therefore, even polymer matrix of CFRP strips melted and decomposed under fire conditions, carbon fibers can hold RC beam and limit its further deflections, as long as anchorage zones of NSM FRP remain intact (Rafi et al. 2008, Ahmed and Kodur 2011). Finally, the unstrengthened RC beam failed at 180 minutes, but NSM FRP strengthened RC beam (Beam I) did not fail for 210 minutes of fire exposure. This also indicated that carbon fibers in Beam I still contribute to load bearing capacity of the beam.

Beam II and the beam tested by Ahmed and Kodur (2011) are FRP strengthened RC beams with U-shaped fire insulation. These two beams were tested under similar fire and loading conditions as shown in Table 4.7. Thus the behavior of these two beams is comparable. It can be seen in Figure 4.15 that deflection response of two beams is very similar, and this is attributed to the fact that both beams were thermally protected and thus steel and FRP retained most of their room temperature strength. External FRP

strengthened RC beam experienced slightly smaller deflection than that of NSM FRP strengthened RC beam. This can be attributed to the fact that relative larger stiffness provided by external FRP laminates than those of NSM FRP strips. Overall, similar response of these two beams also proves the reasonability of fire test results.

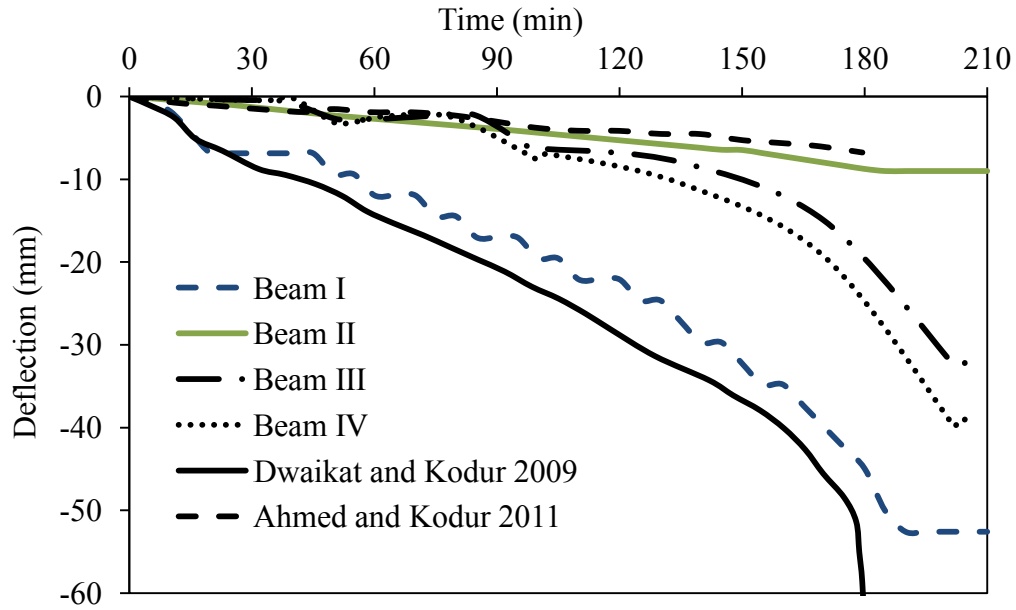


Figure 4.15 Comparison of mid-span deflections of NSM FRP strengthened RC beams with unstrengthened RC beam and external FRP strengthened RC beam

4.6.3.2 Axial restraint force

In the second fire test, axial restraint force and axial displacement at beam support (Beam III) were recorded during the fire test, in order to quantify the influence of axial restraint to fire response of NSM FRP strengthened RC beam. It can be seen in Figure 4.15 that fire induced axial force gradually increased with fire exposure time, and this is mainly due to fire induced expansion of strengthened beam against axial restraint. At later stages of fire exposure, the measured axial restraint force decreased slightly with fire

exposure time. This is mainly attributed to the fact that the beam was slightly detached with the restraint device resulting from relatively larger beam deflection at last stage of fire exposure. However, the effect of axial restraint is still demonstrated through comparative behavior of Beam III and Beam IV. Beam III, which has axial restraints at two ends, achieved smaller deflection and the insulation layer remained attached to beam substrate throughout fire test. While the simply supported Beam IV experienced relatively larger deflection, and this leads to a piece of insulation falling off at later stage of fire exposure.

The variation of axial displacement at the beam support (Beam III) with temperature is also plotted in Figure 4.16. It can be seen that the axial displacement gradual increased with fire exposure time, which indicates that beam slightly expanded in axial direction. Based on the measurement of axial restraint force and displacement, the axial restraint stiffness can be estimated to be 5-10 kN/mm, which is similar to the axial restraint encountered in beam-column frame in buildings.

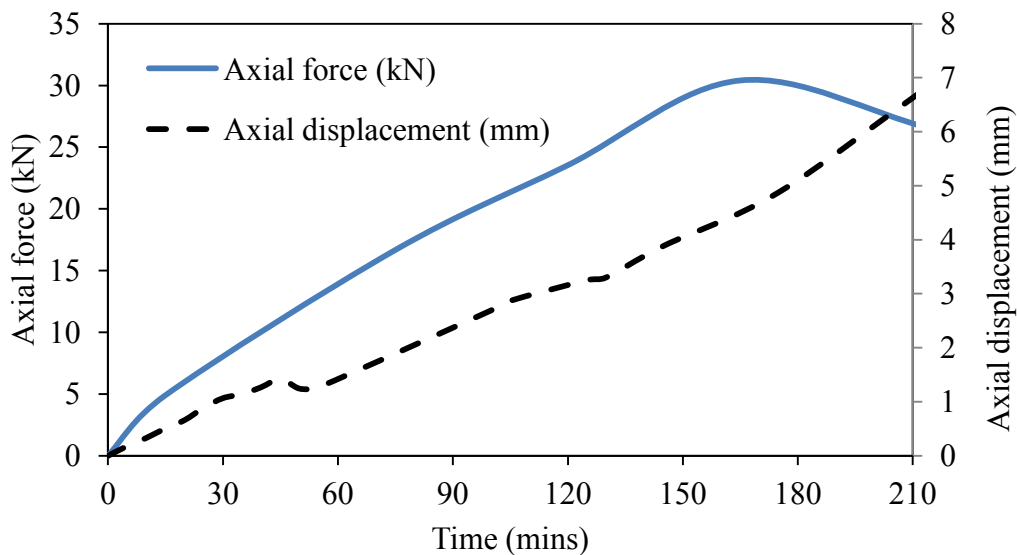


Figure 4.16 Variation of axial force and displacement with fire exposure time

4.6.3.3 Strain in longitudinal reinforcement

The strains in longitudinal reinforcement of the tested beams were monitored utilizing conventional strain gauges since the pre-loading stage. The measured strain at central section of four tested beams is plotted as a function of time in Figures 4.17 and 4.18. It can be seen there is slightly irregular variation of strains measured in the compression and tension rebars, and this is probably due to incremental loading in the pre-loading stage. After 20 to 30 minutes, the tension and compression strains gradually became stable. It can be seen that Beams III and IV attained relatively higher strain levels than those of Beams I and II, due to higher loads applied on the beams. However, for the beams under the same loading level, the strains reached similar values, which also proved effectiveness of strain gauge data.

The strain gauges installed in the beam are regular strain gauges, and they are only functional under 80°C. After fire tests started around 10-15 minutes, all the strain gauges were damaged. Thus strain data in Figure 4.17 stopped after about 40 minutes.

4.6.4 Fire resistance

Time to reach failure under fire exposure is defined as the fire resistance of a structural member. In this experimental program, strength and deflection limit specified in current design standards (ASTM 2012, BSI 2009) were applied to determine failure of beam. According to these limiting criteria, all four NSM FRP strengthened beams did not fail for 210 minutes of fire exposure. This infers these NSM FRP strengthened RC beams possess at least three hours of fire resistance under ASTM E119 standard fire.

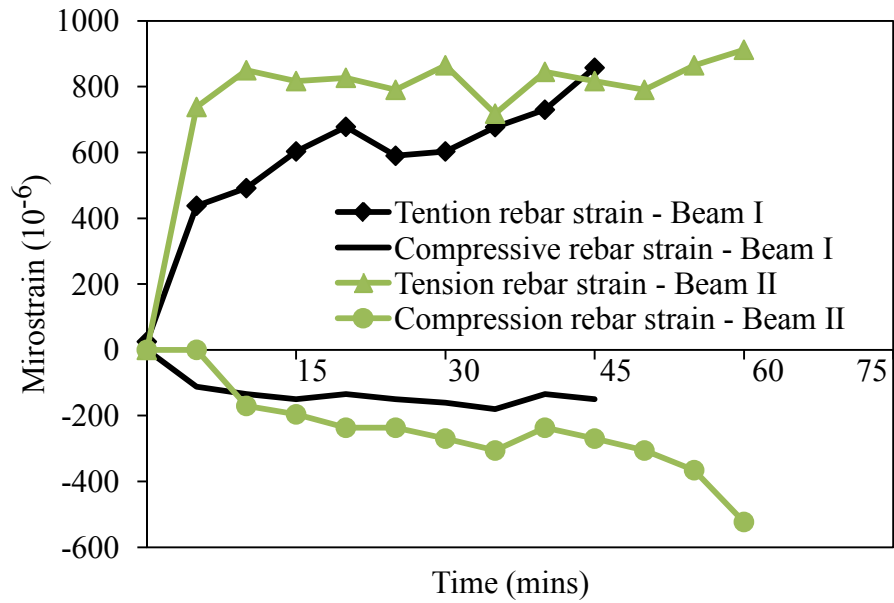


Figure 4.17 Strain measured in tension and compression rebars in Beams I and II during the test (starting from pre-loading stage)

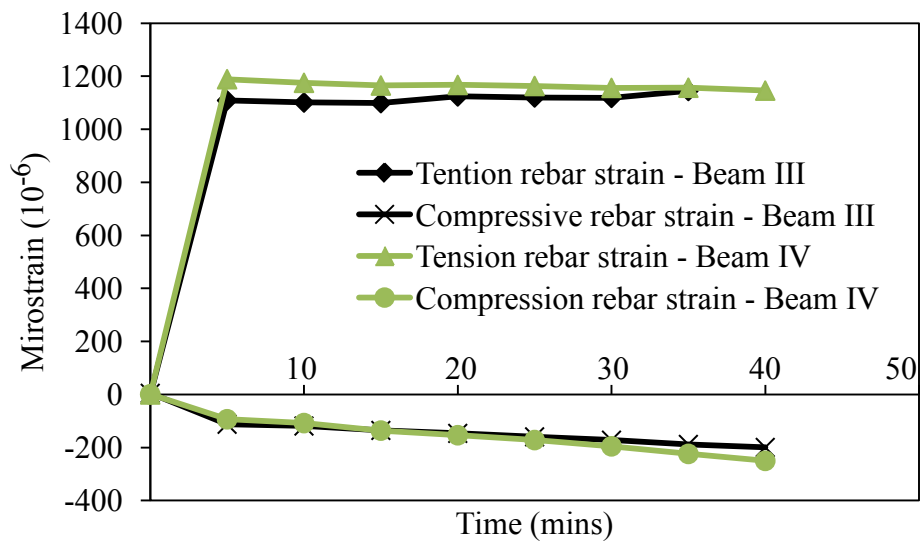


Figure 4.18 Strain measured in tension and compression rebars in Beams III and IV during the test (starting from pre-loading stage)

In current design provisions, the fire resistance of FRP strengthened RC beams is considered to be the same as that of original unstrengthened RC beam, and the contribution of FRP strengthening is usually neglected if no fire insulation is provided (ACI 440.2 2008, FIB Bulletin 14 2007). However, in these fire experiments, although Beam I was unprotected and NSM epoxy experienced severe burning, carbon fibers still provided tensile strength to the beam through “cable action”. As a comparison, an unstrengthened RC beam with similar configuration and load level failed in 180 minutes (Dwaikat and Kodur 2009). This indicates that anchorage zone is vital to achieve relatively higher fire resistance in NSM FRP strengthened RC beams. As long as anchorage zone remains intact, NSM FRP can still contribute to moment capacity even when the beam is not insulated. On the other hand, the axial restraint, which represents typical boundary conditions in buildings, is proved to be beneficial to fire resistance of NSM FRP strengthened RC beams in the fire test. The beam with axial restraint retains relatively smaller deflections, and cracking in the insulation is also reduced. Considering the above factors, it is likely that NSM FRP strengthened RC beams possess satisfactory fire resistance for building applications.

4.7 Residual Strength Tests of NSM FRP Strengthened RC Beams

Since all four tested NSM FRP strengthened RC beams did not fail in the fire resistance tests, all four beam specimens were utilized to study the residual strength of these beams. The test is aiming at evaluating the degradation of load bearing capacity of NSM FRP strengthened RC beams after fire exposure. Details on procedure and results of residual strength test are presented in the following sections.

4.7.1 Test procedure

For the residual strength test, fire exposed Beams I, II and III were allowed to cool down for 24 hours. However, Beam IV was loaded to failure right after 210 minutes of fire exposure, in order to evaluate the residual flexural capacity of fire exposed beams prior to cooling. Beams I, II and IV were tested under simply supported conditions. However, since Beam III was axially restrained during the fire test, and this axial restraint was also applied in residual strength test. The variables studied in residual strength are tabulated in Table 4.8.

In the residual strength test, each beam was loaded to failure under two-point loading (as in fire tests), and the load was increased gradually at 5 kN/min until failure occurred. Similar to the fire tests, the displacements at mid-span and loading points were recorded throughout the loading range.

Table 4.8 Test variables and results in residual strength tests on fire exposed beams

Specimen	Insulation	Boundary condition	Cooling time	Failure load	Failure mode
Beam I	No	Simply supported	24 hours	113 kN	Yielding of steel rebar
Beam II	U-shaped	Simply supported	24 hours	129 kN	Crushing of top concrete
Beam III	U-shaped	Axially restrained	24 hours	132 kN	Crushing of top concrete
Beam IV	U-shaped	Simply supported	No cooling	123 kN	Crushing of top concrete

4.7.2 Results and discussion

The measured load-deflection response for four tested beams is plotted in Figure 4.19. All four beams exhibit similar load-deflection response. The load-deflection response is linear till almost full capacity is attained, and then follows a long plateau

stage. This behavior is more similar to the response of an unstrengthened RC beam, rather than a NSM FRP strengthened RC beam, which has an obvious increase in load capacity after steel rebar enters yielding stage. This load-deflection response indicates NSM FRP strengthening has lost its effectiveness when exposed to fire, mainly resulting from temperature induced bond degradation at FRP-concrete interface. However, a close examination shows that Beams II and III achieved higher residual strength (129 kN and 132 kN) than that of Beam I (113 kN), and this indicates that NSM FRP in Beams II and III contributes to a limited extent to flexural strength capacity of the beam. This is mainly due to the fact that bond at FRP-concrete interface was not completely lost in insulated beams due to relatively low temperatures in NSM FRP.

Beam IV was loaded to failure right after 210 minutes of fire exposure without any cooling, and the residual capacity is 123 kN. This residual strength capacity is lower than that in Beam II, which was tested after full cooling. Also, the stiffness of Beam IV is slightly smaller than those of Beams II and III as shown in Figure 4.19. As discussed in thermal response section, temperatures in steel rebars in Beam IV reached about 350°C. At this temperature level, modulus of elasticity of steel rebars already has about 20% degradation based on temperature-property relations specified in Eurocode 2 (2004), so the stiffness of the whole beam also decreased in some extent correspondingly. While Beam II experienced one day cooling and temperatures in steel rebars were low. Thus Beam II exhibited relatively larger stiffness as compared to that of Beam IV in residual strength tests.

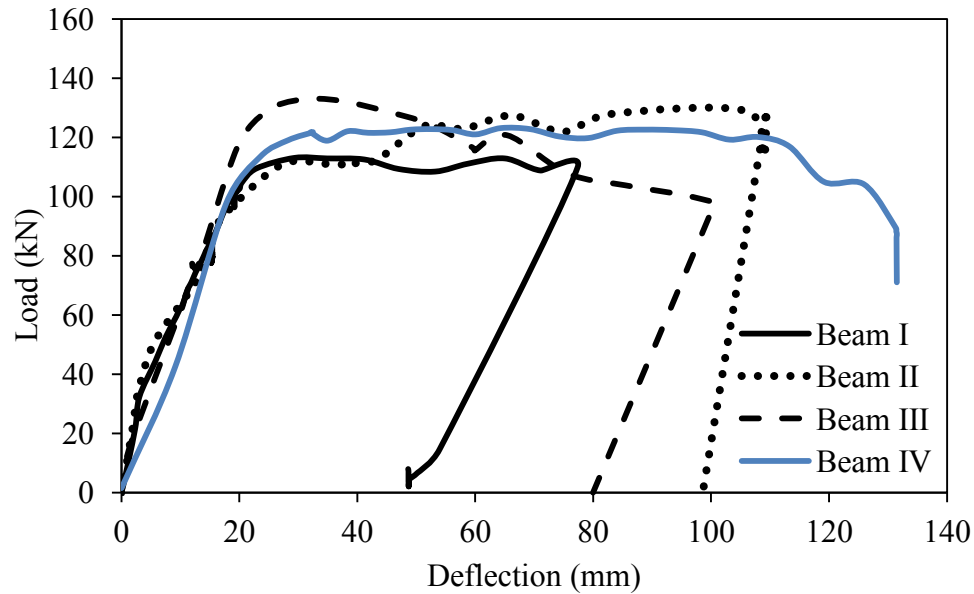


Figure 4.19 Load-deflection response of Beams I-IV in residual strength tests

The failure patterns of four NSM FRP strengthened beams in residual strength tests are illustrated in Figure 4.20. It can be seen that Beam I failed due to yielding of steel reinforcement, which is a typical failure mode in unstrengthened RC beams. However, Beams II, III and IV failed due to crushing of top concrete, and this is more like a failure occurred in NSM FRP strengthened RC beams. A closer observation indicates that major part of FRP strips in Beams II-IV still stayed inside of NSM grooves, as shown in Figure 4.21. This also indicated that NSM FRP in Beams II to IV still possess some strengthening effect even after 210 minutes of fire exposure.



(a) Beam I



(b) Beam II



(c) Beam III



(d) Beam IV

Figure 4.20 Failure patterns of Beams I-IV in residual strength tests



(a) Beam I



(b) Beam II



(c) Beam III



(d) Beam IV

Figure 4.21 Response of NSM FRP strips after failure in residual strength tests

4. 8 Summary

Fire resistance tests were carried out on four NSM FRP strengthened RC beams of T cross section. Three of these strengthened beams were provided with fire insulation, while one beam was tested without any fire insulation. Besides visual observations, temperature and deflection responses were recorded to study fire response of NSM FRP strengthened RC beams. The observations and recorded data allowed to evaluate

comparative performance of NSM FRP strengthened RC beams under different loading and boundary conditions.

All four tests beams did not fail after 210 minutes of fire exposure, judging from strength and deflection criteria. One of these NSM FRP strengthened RC beams was without any fire insulation, and it also did not fail for 210 minutes, indicating NSM FRP strengthened RC beam can achieve sufficient fire resistance through proper design. In addition, comparison of tested beams indicates that presence of fire insulation and axial restraint significantly enhances fire resistance of NSM FRP strengthened RC beams. Results from these fire tests provide a better understanding on the behavior of NSM FRP strengthened RC beams under fire conditions. The data generated from these tests is further utilized for validating numerical models developed for tracing the fire response of NSM FRP strengthened RC beams.

CHAPTER 5

NUMERICAL MODEL

5.1 General

Undertaking fire tests is usually expensive and time consuming. In lieu of fire tests, numerical approach can be applied to evaluate the behavior of a structural member under fire exposure. There have been no numerical studies for evaluating fire response of NSM FRP strengthened RC beams. A sophisticated numerical model, based on finite element approach, is critical for undertaking detailed fire resistance studies on NSM FRP strengthened RC beams. Such a model should account for high temperature properties of constituent materials, various strain components, fire induced bond degradation, and realistic loading and boundary conditions.

A numerical model, initially developed for tracing the fire behavior of RC beams externally strengthened with FRP (Kodur and Ahmed 2010), is extended to simulate the fire performance of RC beams strengthened with NSM FRP reinforcement. The updated model is capable of evaluating fire response of RC beams with various cross-sections, FRP strengthening and insulation schemes. Details of this numerical model are presented in this chapter.

5.2 Macroscopic Finite Element Model for Fire Resistance Analysis

The behavior of RC beams exposed to fire can be simulated using general purpose finite element software such as ANSYS or ABAQUS. In these microscopic finite element

based models, a structural member is generally discretized into a three dimensional mesh, and coupled (or uncoupled) thermal and structural analyses are carried out to trace the fire response of structural members. However, such an analysis is highly complex and requires use skills for discretizing and analyzing results. Also, most of the commercial finite element programs do not account for various strain components (such as creep strain and transient strain in concrete) as well as temperature induced bond degradation at FRP-concrete interface.

As an alternative, macroscopic finite element approach can be used for tracing the fire behavior of RC structural members. In macroscopic finite element model, a sectional analysis is carried out at a number of cross-sections along the length of the member and moment-curvature relationships are generated for these sections to trace the behavior of structural member under a given fire exposure and loading condition. Recently, such macroscopic computer models have been successfully applied to evaluate fire response of RC beams with various configurations (Kodur and Dwaikat 2008, Kodur and Ahmed 2010). In this chapter, such a macroscopic model is further extended to account for the features of NSM FRP strengthened RC beams.

5.2.1 General approach

The numerical model presented here is based on macroscopic finite element approach, and utilizes sectional moment-curvature relationships to trace the response of NSM FRP strengthened RC beams from pre-loading stage to collapse under fire conditions. In the model, an RC beam is discretized into a number of segments along its length, and cross-sectional area of each segment is subdivided into a number of elements,

as shown in Figure 5.1. The mid-section of each segment is assumed to represent the overall behavior of the segment. Fire resistance analysis is carried out by incrementing time in steps. At each time step, the analysis is performed through three stages, namely, (1) evaluating fire temperatures, (2) carrying out heat transfer analysis to determine temperature distribution along cross-section, and (3) conducting structural analysis to determine moment capacity and deflection of beam. The analysis is carried out at various time increments till failure occurs in the beam under any given fire exposure and loading conditions. A flowchart illustrating the steps associated in the model for fire resistance analysis of NSM FRP strengthened beam is presented in Figure 5.2.

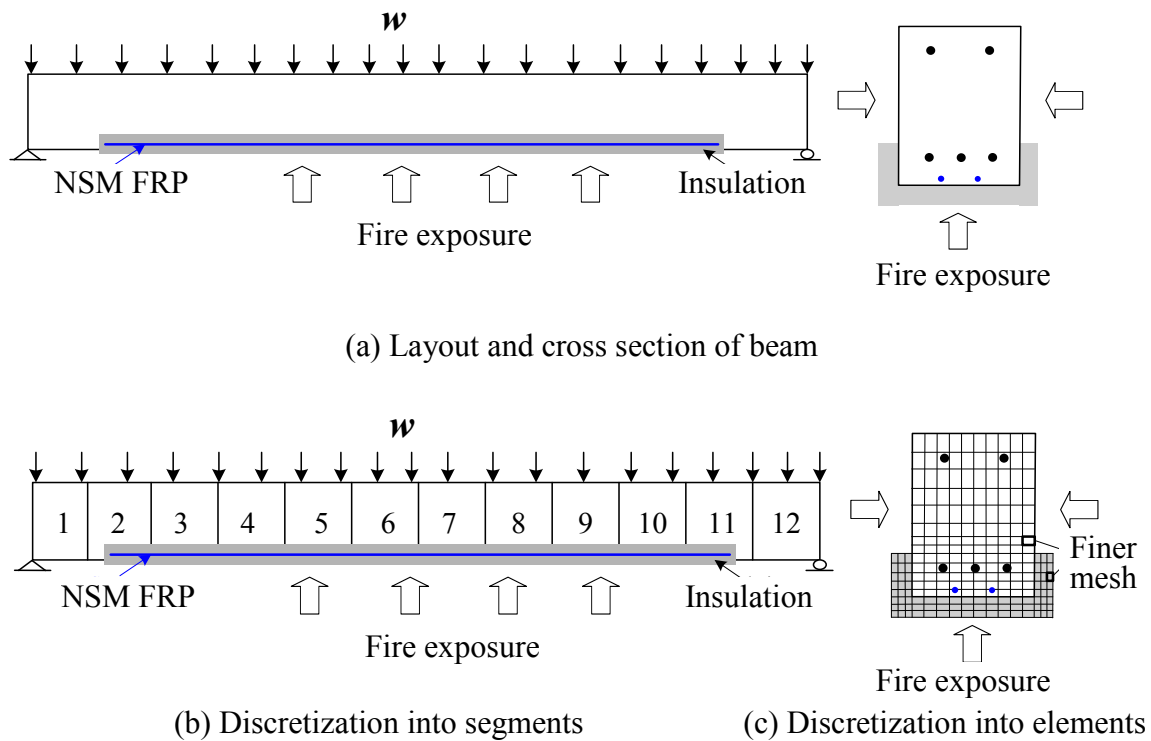


Figure 5.1 Typical beam layout and discretization of beam into segments and elements

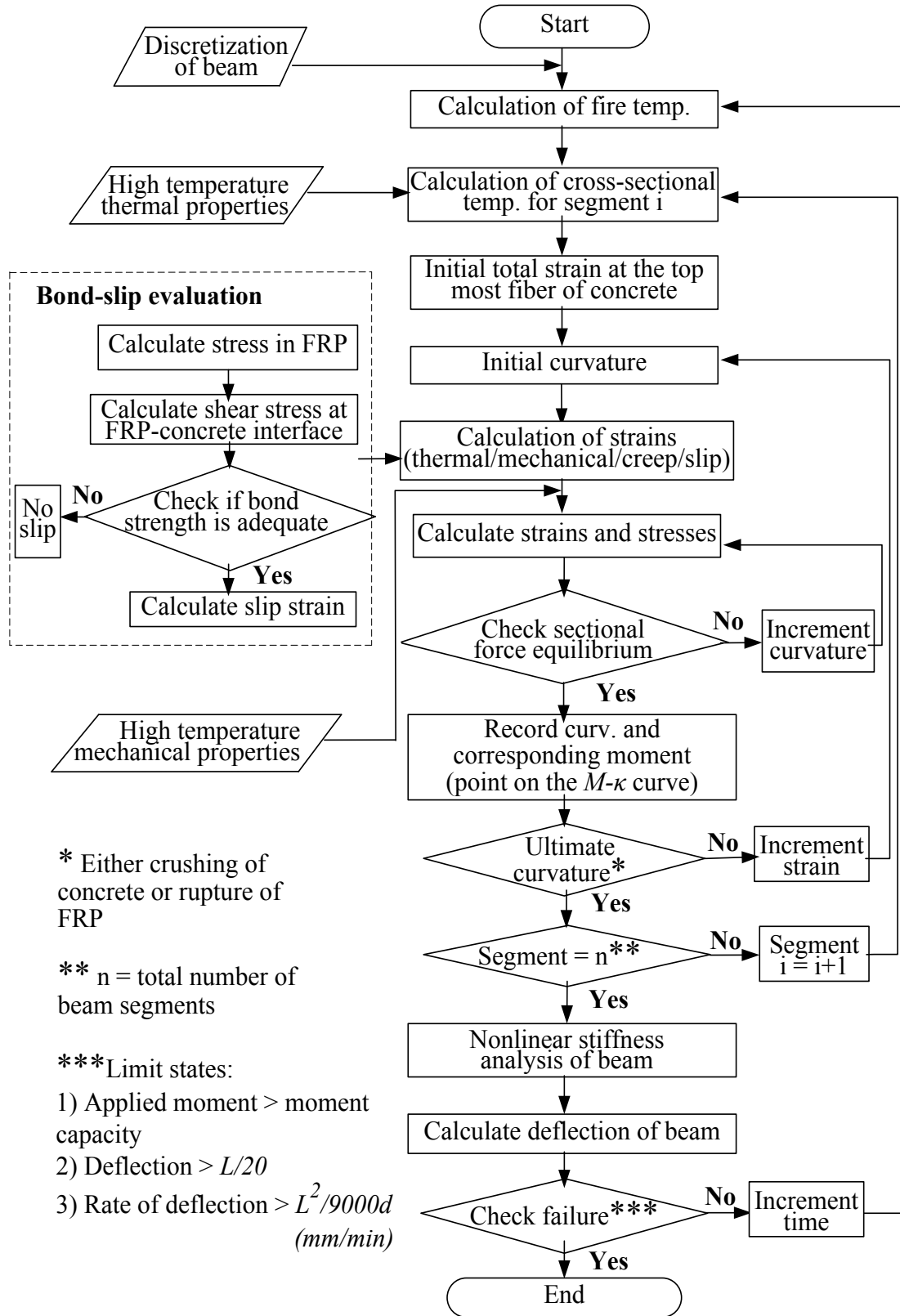


Figure 5.2 Flowchart illustrating the steps associated in the numerical model

5.2.2 Fire temperatures

The fire temperature in the model is evaluated following a standard fire curve such as ASTM E119 (2012) and ISO 834 (2012) fire, or a design fire curve based on specific compartment characteristic. The time-temperature relations for ASTM E119 (2012) and ISO 834 (2012) standard fires can be calculated using the following equations:

$$\text{ASTM E119: } T_f = T_o + 750(1 - e^{-3.79533\sqrt{t}}) + 170.41\sqrt{t} \quad (5.1)$$

$$\text{ISO 834: } T_f = T_o + \log_{10}(8t + 1) \quad (5.2)$$

For a design fire, a decay phase follows after reaching the peak value of fire temperature, and the decay rate mainly depends on a number of factors such as material properties of fuel, size of ventilation, and thermal properties of lining material (Buchanan 2002). User can define any time-temperature relations based on a specific compartment, or utilize design fires specified in current Eurocode 1 (2002) or SFPE Handbook (2008).

5.2.3 Thermal analysis

Thermal analysis on an NSM strengthened RC beam is carried out utilizing finite element approach. The given beam is discretized into a number of segments along beam length, and the central section of the segment, which is assumed to represent the behavior of each segment, is further divided into number of elements. as depicted in Figure 5.1. A finer mesh is applied in the vicinity of critical zones (steel rebar, NSM FRP, and insulation) along the beam cross section to achieve better accuracy in the numerical analysis.

Knowing fire temperatures at various time steps, two-dimensional heat transfer analysis is carried out to evaluate cross-sectional temperature in each segment. It is assumed that the beam is exposed to fire from three sides (bottom and two side surfaces), and fire temperature is uniform along the length of segment. Thus the calculation is performed over a unit length of each segment. Steel reinforcement is not specifically considered in the thermal analysis because it does not influence the temperature distribution in the beam cross section (Lie and Irwin 1993).

Based on the conservation of energy, the governing equation for heat transfer within the beam cross section is given as:

$$k\nabla^2 T + Q = \rho c \frac{\partial T}{\partial t} \quad (5.3)$$

where, k is the thermal conductivity, ρc is the heat capacity, T is the temperature, t is the fire exposure time; and Q is internal heat generation.

It has been established that heat transfer occurring from fire to the surface of beam is through convection and radiation (Buchanan 2002). Conduction is the predominant heat transfer mechanism within the beam. The heat flux due to convection and radiation on the fire exposed surfaces can be evaluated through the following equations:

$$q_{rad} = h_{rad}(T - T_E) \quad (5.4)$$

$$q_{con} = h_{con}(T - T_E) \quad (5.5)$$

where, q_{rad} and q_{con} represent radioactive and convective heat fluxes respectively, and h_{rad} and h_{con} represent radioactive and convective heat transfer coefficient respectively.

T_E is external temperature surrounding the boundary. Hence the total heat flux on the beam boundary (q_b) can be given by the following equation:

$$q_b = (h_{con} + h_{rad})(T - T_E) \quad (5.6)$$

Using Fourier's Law, the governing heat transfer equation on the boundary of the beam can be written as:

$$k \left(\frac{\partial T}{\partial y} n_y + \frac{\partial T}{\partial z} n_z \right) = -h(T - T_E) \quad (5.7)$$

where, n_y and n_z are components of the vector normal to the boundary in the plane of the cross-section, and

$$h = h_{con} + h_{rad} \quad (5.8)$$

The beam is exposed to fire from bottom and two side surfaces, while the top surface of beam remains cool. Thus two types of boundary conditions should be considered for thermal analysis, namely:

- 1) Fire exposed boundaries where the heat flux is governing by:

$$k \left(\frac{\partial T}{\partial y} n_y + \frac{\partial T}{\partial z} n_z \right) = -h_f(T - T_f) \quad (5.9)$$

- 2) Unexposed boundaries where the heat flux is governing by:

$$k \left(\frac{\partial T}{\partial y} n_y + \frac{\partial T}{\partial z} n_z \right) = -h_c(T - T_0) \quad (5.10)$$

where, h_f and h_c are heat transfer coefficient of the fire side and room temperature side respectively, and T_f and T_0 are fire and room temperature respectively.

Galerkin finite element formulation is applied to solve Eq. 5.3. In this formulation, the material property matrices (stiffness matrix K_e , mass matrix M_e) and the equivalent nodal heat flux (F_e) are generated for each element. These matrices are given by following equations (William 1990):

$$K_e = \int_A k \left[\frac{\partial N}{\partial x} \frac{\partial N^T}{\partial x} + \frac{\partial N}{\partial y} \frac{\partial N^T}{\partial y} \right] dA + \int_{\Gamma} N \alpha N^T ds \quad (5.11)$$

$$M_e = \int_A \rho c N N^T dA \quad (5.12)$$

$$F_e = \int_A N Q dA + \int_{\Gamma} N \alpha T_{\alpha} ds \quad (5.13)$$

where, N is the vector of the shape functions, k is the thermal conductivity, α is h_c or h_f depending on the boundary condition Γ , Q is heat source; s is distance along the boundary, A is the area of the element, and T_{α} is fire or ambient temperature depending on boundary condition Γ .

Once the matrices of elements are computed, they are assembled into a global system of differential equations which is expressed as:

$$M\dot{T} + KT = F(t) \quad (5.14)$$

where, K is global stiffness matrix, M is global mass matrix, F is equivalent nodal heat flux, and $\dot{T}$ is temperature derivative with respect to time (t).

The above equation is solved using finite difference algorithm of trapezoidal family (θ algorithm) in the time domain. This algorithm computes temperature distribution at any time step (n+1) using the information available at preceding time step (n), and can be written as (William 1990):

$$T_{n+1} = T_n(\theta \dot{T}_{n+1} + (1-\theta)\dot{T}_n) \quad (5.15)$$

Multiplying both side of Eq. (5.15) by M and using Eq. (5.14) at the beginning and the end of the time interval (t_n, t_{n+1}) , the following equation can be obtained:

$$(M + h\theta K)T_{n+1} = (M - h(1-\theta)K)T_n + h(\theta F_{n+1} + (1-\theta)F_n) \quad (5.16)$$

where, h is time step, T_n and T_{n+1} are temperatures at the beginning and the end of time step respectively. F_n and F_{n+1} are the equivalent nodal heat flux at the beginning and the end of time step, and θ is a constant between 0 and 1.

For unconditional stability of numerical calculations, θ has to be equal to or greater than 0.5 (William 1990). By knowing the temperatures at ambient conditions, Eq. 5.16 can be applied to obtain the time history for temperature at the following time step, and this can be repeated for subsequent time steps. In each time step, an iterative process is required to solve Eq. 5.16 due to the nonlinearity of both material properties and boundary conditions. More details on the finite element formulation for solving the heat and mass transfer equations are provided in Appendix C. The obtained nodal temperature from Eq. 5.16 is utilized to calculate the temperature in each element by averaging the four nodal temperatures of rectangular elements. For steel rebars the temperature is assumed to be that at the center of the rebar.

5.2.4 Structural analysis

5.2.4.1 General analysis procedure

After generating cross sectional temperature of the beam at various time steps, structural analysis is performed to evaluate the fire resistance of NSM FRP strengthened

RC beams. In structural analysis, the calculation is conducted using the same mesh as used for thermal analysis. The strains and stresses in each element are represented by the corresponding values at the center of the element. The temperatures in each element obtained from thermal analysis are used as input, and segmental $M-\kappa$ relationships are developed to trace the structural behavior of NSM FRP strengthened RC beams at various time steps. The structural analysis proposed here is based on the following assumptions.

- Plane sections before bending remain plane after bending.
- Beam has a constant cross-section.
- The failure of beam is through flexural strength limit and the beam does not fail in shear strength limit.
- There is no bond-slip between steel reinforcement and concrete at various temperatures.
- FRP reinforcement exhibits linear stress-strain relationship at various temperatures up to failure.

At each time step, the structural analysis is performed by first estimating fire induced axial force and slip at NSM FRP/concrete interface in each beam segment. Then $M-\kappa$ relationship for each beam segment is generated based on force equilibrium and strain compatibility principles. At last, nonlinear stiffness analysis is conducted to evaluate the structural response (moment capacity, deflection, stress) of NSM FRP strengthened RC beam at each time step. Details on structural analysis and calculations are described in the following sections.

5.2.4.2 Evaluating temperature induced slip and axial restraint force

In NSM FRP strengthened RC beam, bond at the interface of NSM FRP, adhesive and concrete plays a critical role in transferring stresses from concrete substrate to FRP reinforcement. With increase of temperature in NSM adhesive, bond properties deteriorate rapidly and this introduces slip at FRP-adhesive-concrete interface. Eventually, the NSM adhesive loses its effectiveness in transferring stresses and thus debonding of NSM FRP occurs.

In the numerical model, temperature induced slip and bond degradation is accounted for introducing a slip strain in NSM FRP reinforcement. When slip occurs, strain resulting from slip is added into the total strain of FRP, and thus the effective mechanical strain (loading resistance) is reduced. To evaluate slip strain at various temperatures, the amount of slip that occurs in each segment is to be calculated. For this a bond-slip relation proposed by Sena Cruz and Barros (2004) is incorporated into the numerical model, and is given by:

$$\tau = \tau_m \cdot \left(\frac{s}{s_m} \right)^\alpha, \quad s \leq s_m \quad (5.17)$$

$$\tau = \tau_m \cdot \left(\frac{s}{s_m} \right)^{-\alpha'}, \quad s > s_m \quad (5.18)$$

where, τ and s represent shear stress and corresponding slip developed at FRP-concrete interface, and τ_m and s_m are peak bond stress and corresponding slip, respectively. α and α' are parameters defining the shape of curves, and their values follow the data reported by Sena Cruz and Barros (2004). τ_m and s_m are a function of temperature, and the

variation of bond strength (τ_m) and corresponding slip (s_m) of NSM FRP reinforcement with temperature is plotted in Figure 5.3, which is obtained from high temperature bond test data presented in Chapter 3.

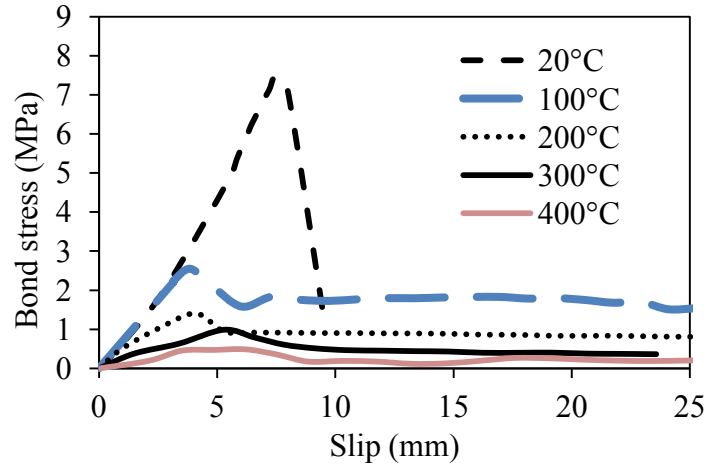


Figure 5.3 Bond stress-slip relations of NSM FRP strip at various temperatures

Equations for evaluating bond-slip in a segment can be derived by applying for equilibrium at the FRP-concrete interface and can be written as:

$$\tau_i(t) \cdot (P_{FRP} \cdot L_i) = (\sigma_{i+1}(t) - \sigma_i(t)) \cdot (A_{FRP}) \quad (5.19)$$

where, $\sigma_i(t)$ is the stress in FRP reinforcement for segment i at time step t , P_{FRP} and A_{FRP} are perimeter and area of FRP reinforcement respectively, L_i is the length of segment i , the shear stress $\tau_i(t)$ is assumed to be uniformly distributed along the segment length, and the average shear stress τ_i at the FRP-concrete interface can be calculated using Eq. 5.19.

Knowing $\tau_i(t)$, the slip of FRP reinforcement at each time step can be evaluated as given in Eqns. 5.17 and 5.18. Then the slip strain in FRP reinforcement can be obtained as:

$$\varepsilon_{slip} = s / L_i \quad (5.20)$$

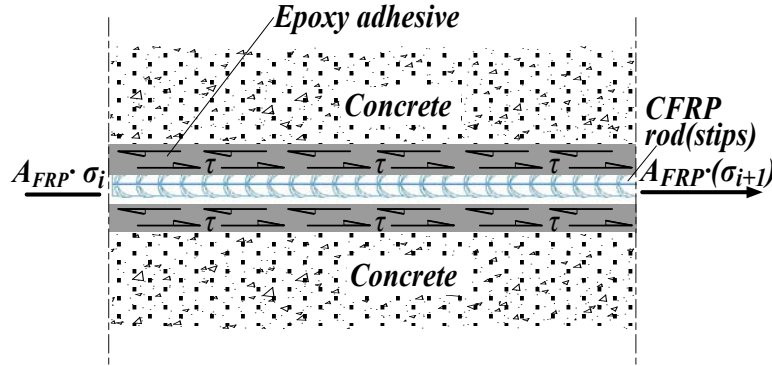


Figure 5.4 Force equilibrium at NSM FRP-concrete interface in the i th segment (vertical view)

In addition to the slip at NSM FRP reinforcement, temperature induced axial restraint force also influences the behavior of NSM FRP strengthened RC beams. This is mainly attributed to the fact that RC beams can experience considerable expansion when exposed to severe temperatures. Generally beams in buildings are restrained by columns or shear walls, and thus significant axial restraint force can develop at the supports under fire conditions. This needs to be properly accounted in the numerical analysis. For this an approach recommended by Dwaikat and Kodur (2008) used for predicting axial forces in RC beam, is applied to determine fire induced axial restraint force in NSM FRP strengthened beams.

The total axial restraint force (P) that develops in the beam can be calculated from summation of compressive and tensile forces in each element of the beam (segment), which is given as:

$$P = T + C = \sum \sigma_m A_m \quad (5.21)$$

where, σ_m is stress at the center of each element, and A_m is area of the corresponding element. Since stress in each element can be computed using a given central total strain and the curvature of beam, the axial force in each beam segment (P_i) can be related to the corresponding central total strain (ε_{0i}) and the curvature (κ_i) as follows:

$$P_i = \phi(\varepsilon_{0i}, \kappa_i^n) \quad (5.22)$$

In this numerical model, axial restraint force in each beam segments is assumed to be constant at a given time step. At the beginning of each time step, the curvature in beam segment, is equal to the curvature computed in the preceding time step ($n-1$), and for a small increment in time step, the difference in curvature is usually very small. With these assumptions, Eq. (5.22) can be expressed in terms of central total strain (ε_0) and curvature (κ) for each beam segment i as

$$P_i = \phi(\varepsilon_{0i}, \kappa_i^n) \approx \phi(\varepsilon_{0i}, \kappa_i^{n-1}) \quad (5.23)$$

The total central strain in each segment is used to check the compatibility conditions, and the value of axial force (P) is modified until compatibility and equilibrium conditions are satisfied. The compatibility conditions along the span of the beam need to be satisfied to the following conditions:

$$\sum l_i - L - \Delta = 0 \quad (5.24)$$

where L is length of the beam, and Δ is total axial expansion in beam length. l_i is projected length of deformed segment i , and it can be calculated as follows.

$$l_i = \sqrt{(s_i)^2 - (w_{i2}^n - w_{i1}^n)^2} \approx \sqrt{(s_i)^2 - (w_{i2}^{n-1} - w_{i1}^{n-1})^2} \quad (5.25)$$

where s_i is the length of deformed segment i , w_{i2}^{n-1} and w_{i1}^{n-1} are deflections at the beginning and the end of beam segment in the $(n-1)^{\text{th}}$ time step, and w_{i2}^n and w_{i1}^n are deflections at the beginning and the end of beam segment in the n^{th} time step, as illustrated in Figure 5.5.

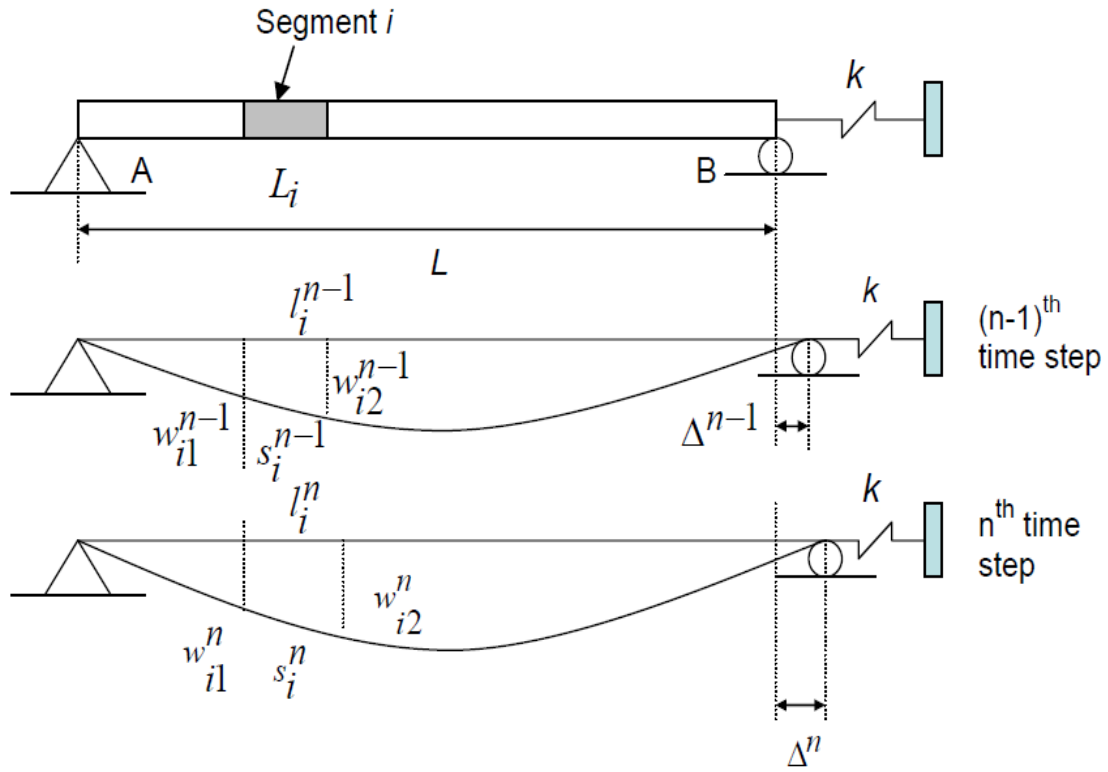
Since $s_i = (1 + \varepsilon_{0i}) L_i$, Eq. 5.24 can be expressed as

$$\sum \sqrt{(1 + \varepsilon_{0i})^2 L_i^2 - (w_{i2}^{n-1} - w_{i1}^{n-1})^2} - L - \Delta = 0 \quad (5.26)$$

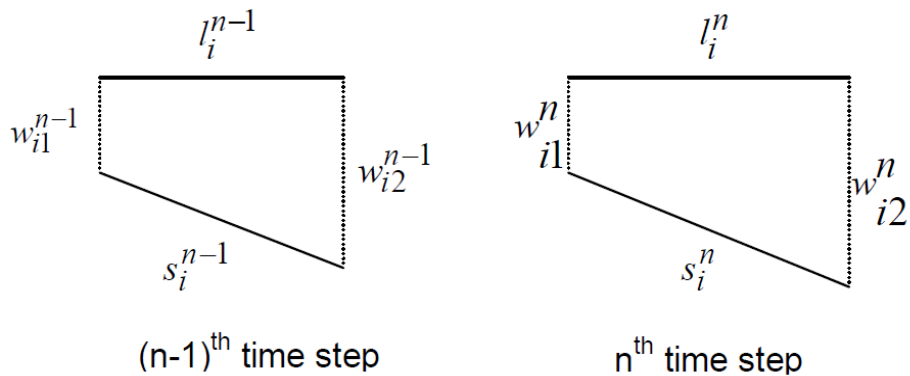
To satisfy both force equilibrium and strain compatibility, the axial restraint force (P) that develops in the fire exposed beam is calculated through the following iterative procedure (Ahmed 2010):

- Assume a value of ' P ' (axial restraint force) for known value of curvature (κ_i^{n-1}) from preceding time step $(n-1)$. In the first time step (at room temperature), $P = 0$.
- Compute central total strain (ε_{0i}) in each segment i of beam.
- Calculate axial displacement (Δ) for a known value of spring stiffness (k).
- Check compatibility using Eq. (5.26)
- Update axial force (P) until Eq. (5.26) is satisfied within a pre-determined tolerance value.

Once the axial restraint force is computed through iterative procedure stated above, $M-\kappa$ relationships are to be generated for further structural analysis. Accounting for fire induced axial restraint force is critical in fire resistance analysis of NSM FRP strengthened RC beams.



(a) Deflected beam at the $(n-1)^{th}$ and n^{th} time steps



(b) Typical beam segment

Figure 5.5 Illustration of axial restraint force calculations

5.2.4.3 Generation of moment-curvature ($M-\kappa$) relationships

After temperature induced slip and axial restraint force in the beam are calculated, the $M-\kappa$ relationships of each beam segment are generated through an approach analogous to analysis of a prestressed concrete beam. The calculation starts with an assumed value of strain at the top fiber of concrete (ε_c) and curvature (κ), and the total strain (ε_t) in each element of concrete, FRP and rebar can be evaluated as:

$$\varepsilon_t = \varepsilon_c + \kappa y \quad (5.24)$$

where ε_t is the total strain in any given element, ε_c is the strain at the top most fiber in concrete, κ is the curvature, y is the distance from the top layer (concrete) to the center of the given element. During fire exposure, a concrete member experiences strain due to thermal, mechanical (loading) and creep effects. In concrete (element), the resulting total strain is the sum of thermal, mechanical, transient and creep strains. However, in reinforcing steel bars, the total strain is the sum of thermal, mechanical and creep strains. In the case of NSM FRP reinforcement, the total strain is the sum of thermal, mechanical, creep and slip strains. Thus, the mechanical strain in each element of concrete, FRP and steel rebar is respectively evaluated as,

$$\varepsilon_{mec} = \varepsilon_{tc} - \varepsilon_{thc} - \varepsilon_{crc} - \varepsilon_{trc} \text{ (in concrete)} \quad (5.25)$$

$$\varepsilon_{mes} = \varepsilon_{ts} - \varepsilon_{ths} - \varepsilon_{crs} \text{ (in steel)} \quad (5.26)$$

$$\varepsilon_{mef} = \varepsilon_{tf} - \varepsilon_{thf} - \varepsilon_{crf} - \varepsilon_{slip} \text{ (in FRP)} \quad (5.27)$$

where ε_{tc} , ε_{thc} , ε_{mec} , ε_{crc} , ε_{trc} represent total, thermal, mechanical, creep, and transient strains respectively in concrete element; ε_{ts} , ε_{ths} , ε_{mes} , ε_{crs} represent total, thermal,

mechanical, and creep strains respectively in steel rebar; and ε_{tf} , ε_{thf} , ε_{mef} , ε_{crf} , ε_{slip} represent total, thermal, mechanical, creep and slip strains respectively in FRP reinforcement. The expressions and associated reference for evaluation of strain components in constituent materials is tabulated in Table 5.1.

The above computed mechanical strains are utilized to obtain stress and force in each element, utilizing relevant time-dependant stress-strain relations of concrete, steel and FRP. These relations are given in Appendix A. An iterative procedure is applied to evaluate the curvature and stress for a given (assumed) concrete strain (ε_c), till force equilibrium and strain compatibility are satisfied, as shown in Figure 5.6. The moment and curvature corresponding to that strain level are computed to represent one point on $M-\kappa$ curve. For each time step, various points on the $M-\kappa$ curve are generated until concrete strain at the top most fiber reaches its limiting (failure) strain. With these time-dependant $M-\kappa$ relationships, the structural behavior of NSM FRP strengthened beam can be further traced through nonlinear beam analysis.

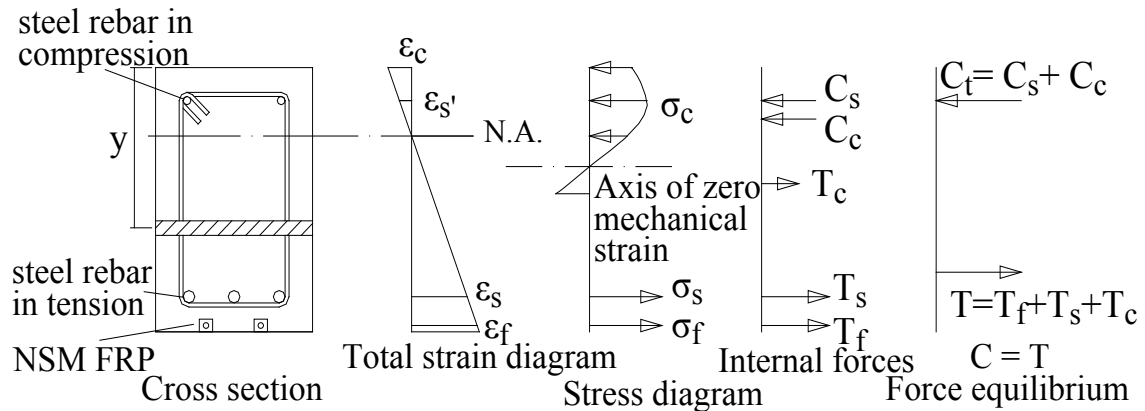


Figure 5.6 Force equilibrium and strain compatibility in an RC beam strengthened with NSM FRP

5.2.4.4 Beam analysis

The $M-\kappa$ relationships, slip in NSM FRP and axial restraint force generated for various segments form the basis for tracing the response of NSM FRP strengthened beam exposed to fire. With time-dependant $M-\kappa$ relationships, the secant stiffness of each segment can be determined based on the moment level reached in that particular segment, and then nonlinear stiffness analysis is performed to evaluate the deflection response of NSM FRP strengthened RC beam.

Each node in the idealized beam is assumed to have two degrees of freedom, namely; rotation and vertical displacement. The deflection of the beam is calculated through a stiffness approach and utilizing an iterative procedure described by Cambell and Kodur (1990). The first step in this stiffness analysis is to apply a unit load to determine the moment and corresponding curvature in each beam segment. The initial stiffness (EI_0), evaluated based on stiffness at elastic condition, is applied in this step as shown in Figure 5.6. The segment with the maximum moment is selected as the critical segment of the beam.

Then a target curvature in the key beam segment is selected on pre-generated $M-\kappa$ curve. Utilizing unit load analysis, a scaling factor is computed by dividing the target curvature with unit load curvature in the key segment. The unit load curvatures in all beam segments are scaled by this scaling factor. Based on scaled curvatures, the secant rigidity can also be updated from segmental $M-\kappa$ relationships. An iterative procedure, illustrated in Figure 5.6, is employed until convergence of segmental secant stiffness is achieved within certain tolerance. Once the tolerance is achieved, the above procedure is repeated for next assumed target curvature (Dwaikat 2009, Ahmed 2010).

Table 5.1 Strain components in concrete, steel, and FRP

Material	Strain component	Expression and reference (source)
Concrete	ε_{mec}	effective strain for resisting external load
	ε_{thc}	$\varepsilon_{thc} = [0.004(T^2 - 400) + 6(T - 20)] \times 10^{-6}$ (Lie 1992)
	ε_{crc}	$\varepsilon_{crc} = \beta_1 \frac{\sigma}{f_{c,T}} \sqrt{t} e^{d(T-293)}$ (Harmathy 1993) $\beta_1 = 6.28 \times 10^{-6} s^{-0.5}$ (constant), $d = 2.658 \times 10^{-3} K^{-1}$ (empirical constant), T = current concrete temperature (K), t = time (s), $f_{c,T}$ = concrete strength at temperature T , and σ = stress in the concrete
	ε_{trc}	$\Delta \varepsilon_{tr} = k_2 \frac{\sigma}{f_{c,20}} \Delta \varepsilon_{th}$ (Anderberg and Thelandersson 1976) k_2 = a constant that ranges between 1.8 and 2.35 ($k_2 = 2$ in the analysis), $\Delta \varepsilon_{th}$ = change in thermal strain, $\Delta \varepsilon_{tr}$ = change in transient strain, $f_{c,20}$ = concrete strength at 20°C
Steel	ε_{mes}	effective strain for resisting external load
	ε_{ths}	$\varepsilon_{ths} = [0.004(T^2 - 400) + 6(T - 20)] \times 10^{-6}$ $T < 1000^\circ C$ (Lie 1992)
	ε_{crs}	$\varepsilon_{crs} = (3Z\varepsilon_{t0}^2)^{1/3} \theta^{1/3} + Z\theta$ (Harmathy 1967) $Z = \begin{cases} 6.755 \times 10^{19} (\sigma / f_y)^{4.7} & \sigma / f_y \leq 5/12 \\ 1.23 \times 10^{16} (e^{10.8(\sigma / f_y)}) & \sigma / f_y > 5/12 \end{cases} \square$ $\theta = \int e^{-\Delta H / RT} dt$, $\Delta H / R = 38900 K$, t = time (hours), $\varepsilon_{t0} = 0.016(\sigma / f_y)^{1.75}$, σ = stress in steel as function of temperature, and f_y = yield strength of steel (room temperature).
FRP	ε_{mef}	effective strain for resisting external load
	ε_{thf}	GFRP: $\varepsilon_{thf} = 10 \times 10^{-6} (T - 20)$ CFRP: 0
	ε_{crf}	$\varepsilon_{crf} = \int_0^t B \sigma^{0.01} e^{-H/kT} \sinh\left(\frac{\sigma}{kT}\right) dt$ $B = 2.03 \times 10^{-4} T^{1.55} \cdot t^{0.25}$ T is FRP temperature (K), σ is the stress in FRP (MPa), t is the fire exposure time (s), and k is Boltzmann's constant. H is the activation energy which follows the reported experimental data
	ε_{slip}	See section 5.2.4.2

After each iteration procedure, stiffness matrix and the loading vector are computed for each longitudinal segment and assembled in the form of a nonlinear global stiffness equation, and solved to compute deflections at that time step:

$$[K_g] \cdot [\delta] = [P] \quad (5.28)$$

where: K_g = global stiffness matrix, δ = nodal displacements.

$$P = P_f + P_s \quad (5.29)$$

where, P_f is equivalent load vector due to applied loading and P_s is equivalent nodal vector due to P effect.

The effect of the second order moments, developed due to the axial restraint force, is calculated using the following equation:

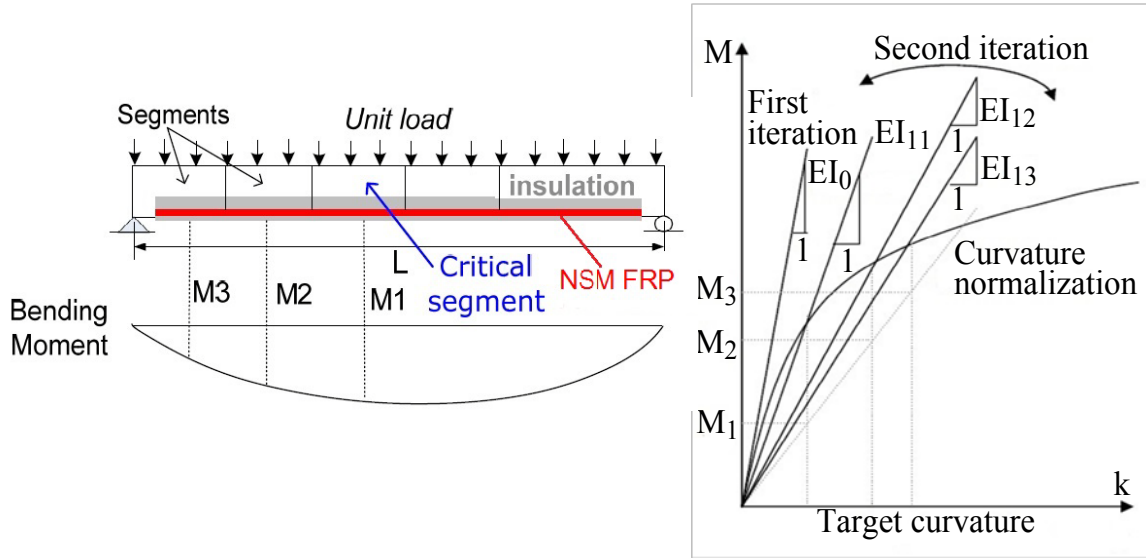
$$[P_s] = -[K_{geo}] [P] \quad (5.30)$$

where, $[K_{geo}]$ is geometric stiffness matrix, $[\delta]$ is nodal displacements, and $[P_s]$ is equivalent nodal load vector due to P - δ effect.

Various response parameters, including temperature and stress in rebar, mid-span deflection, and moment capacity, generated from the program, are utilized to evaluate failure of the beam at each time step. The time step at which failure of the beam is attained is taken to be the fire resistance of the beam. The beam is said to attain failure when any of the below limiting criterion is reached:

1. The moment due to applied service load exceeds the capacity of the beam (ASTM, 2012).

2. The deflection of the beam exceeds $L/20$ (where L is the length of beam) at any fire exposure time (BS 476, 1987).
3. The rate of deflection exceeds the limiting deflection rate $[L^2/9000d \text{ (mm/min)}]$, where d is the effective depth of beam] (BS 476, 1987).



(a) Segments in idealized beam and bending moment diagram (b) M - κ of beam segment

Figure 5.7 Illustration of curvature controlled iterative procedure for beam analysis

5.3 Computer Implementation

The macroscopic finite element model described in above is developed into an executable computer program using FORTRAN language. In this section, input and output data as well as material properties used in this program are discussed in detail.

5.3.1 Input data

The input data for this numerical program comprises of four components, namely, geometric properties, material properties, fire and loading conditions, and analysis options.

- Geometric properties: mainly refer to elevation and cross-sectional dimension of beam, locations of steel and FRP reinforcement, and layout and thickness of insulation.
- Material properties: include thermal and mechanical properties of constituent materials, such as concrete, reinforcing steel, FRP, and insulation. The variation of thermal and mechanical properties of these materials with temperature can be input as a set of factors normalized to the room temperature values.
- Fire, loading and boundary conditions: For fire exposure, user can select a standard fire or design fire incorporated in the program, or define a specified time-temperature curve. The available loading conditions include uniformly distributed load and concentrated load (one-point or two-point loading) on the beam. Two types of boundary conditions, simply supported and axially restrained, can be applied to the beam. The axial restraint stiffness at beam support can be defined based on specific boundary situations.
- Analysis options: mainly used for controlling the convergence and accuracy of the analysis. The available options include curvature increment, criteria of convergence, and criteria of beam failure.

The sequential order of the input data must be followed in the input file. Consistent SI units are used throughout the input file.

5.3.2 Output results

Thermal and structural response parameters of NSM FRP strengthened RC beam are output at each time step. Thermal response parameters include cross sectional temperatures in concrete, steel, and FRP. Structural response mainly includes moment-curvature relationship strength capacity, deflection, etc.

- Thermal response: temperature in each element is output at each time step. In addition, temperatures at critical locations, like reinforcing steel, FRP, center of concrete, are output in separated files.
- Moment-curvature relationship: $M-\kappa$ relationships in each segment of beam are output at various time steps.
- Moment capacity and deflection: remaining moment capacity of the beam and mid-span deflections at each time step are generated in output files. The program automatically stops when the beam reaches failure limit. Thus the time of final analysis step is taken as the fire resistance of beam.
- Stress: the stresses in each steel and FRP reinforcement are generated at each incremental time step. This information can be used to evaluate strength contribution from steel and FRP reinforcement under fire exposure.

The output results are also in SI units, and each component of output results (temperatures, strength capacity, etc) are stored in separated files for ease of data analysis.

5.3.3 Material properties

High temperature properties of constitutive materials have significant influence on thermal and structural response of concrete member exposed to fire. For concrete,

high-temperature thermal and mechanical property relations as per Eurocode (2004) and Lie (1992) provisions are incorporated in the numerical model, and user can select appropriate properties according to specified concrete strength (normal strength or high strength) and aggregate type (carbonate or siliceous). The spalling of concrete is not specifically considered in this model, since FRP strengthening is generally applied on concrete structures made of normal strength concrete (NSC), and fire induced spalling is usually not a major concern in NSC beams (Kodur and Phan 2007).

For steel reinforcement, only high-temperature mechanical properties are accounted for in the analysis. Relevant relations for temperature dependant stress-strain curves for reinforcing steel, taken from Lie (1992) and Eurocode (2004) standards, is built into the computer model. Thermal properties of steel rebar are not specifically considered since they do not significantly affect temperature distribution within the beam cross section (Lie and Irwin 1993, Kodur and Dwaikat 2008).

Since temperature in epoxy adhesive has critical influence on bond degradation of NSM FRP, thermal properties of epoxy are incorporated into the numerical model. The input values of thermal properties of epoxy are determined based on the data reported in literatures (Chern et al. 2002, Shokralla and Al-Muaiikel 2009, Kandare et al. 2010). Temperature-dependant thermal properties of epoxy are provided in Appendix A.

For NSM FRP, it has been established that FRP follows a linear stress-strain response both at room temperature (ACI 440.1 2006, FIB 40 2007) and at elevated temperatures (Bisby 2003, Wang et. al 2007). Thus, temperature dependent stress-strain response of FRP can be represented through a set of linear relationships. The peak value

(tensile strength) and slope (elastic modulus) of these linear relations are taken from the test data generated in Chapter 3.

The temperature-dependant thermal properties of insulation are also built into the numerical model. The relations for thermal conductivity and thermal capacity of fire insulation follow the test data reported by Bisby (2003).

All the temperature dependant thermal and mechanical properties of constitutive materials, including concrete, reinforcing steel, FRP, epoxy, and insulation, are presented in Appendix A.

5.4 Validation of Numerical Model

The validity of the above model is established by comparing predictions from the model with measured response parameters in experiments both at ambient and fire conditions. The tested beams used in validation are selected from literature (Rasheed et al. 2010, Palmieri 2012) and from fire resistance tests carried out as part of this dissertation. The response parameters covered in validation include load-displacement response at ambient conditions, cross-sectional temperatures, mid-span deflections, and fire resistance of beam. The detailed validation process is described below.

5.4.1 Response at ambient conditions

For room temperature validation, three RC beams (designated as Beam V1, V2 and V3) tested by Rasheed et. al (2010) were analyzed using the above developed model. The selected beams are of 254×457 mm rectangular cross-section with an effective span of 4.72 m. Each beam is divided into 40 segments, and the cross-section of each segment

is subdivided into 800 elements of 12×11 mm (similar to that in Figure 5.1). The main flexural reinforcement in the beam comprise of four ϕ 19 mm steel rebars, while two ϕ 9 mm steel rebars form the compressive reinforcement. Beam V1 is a conventional RC beam (no strengthening), Beam V2 is an RC beam strengthened with eight 16×2 mm NSM CFRP strips (four grooves, two strips per groove), and Beam V3 is an RC beam externally strengthened with CFRP laminates. Details of beam configuration and material properties are shown in Table 5.2.

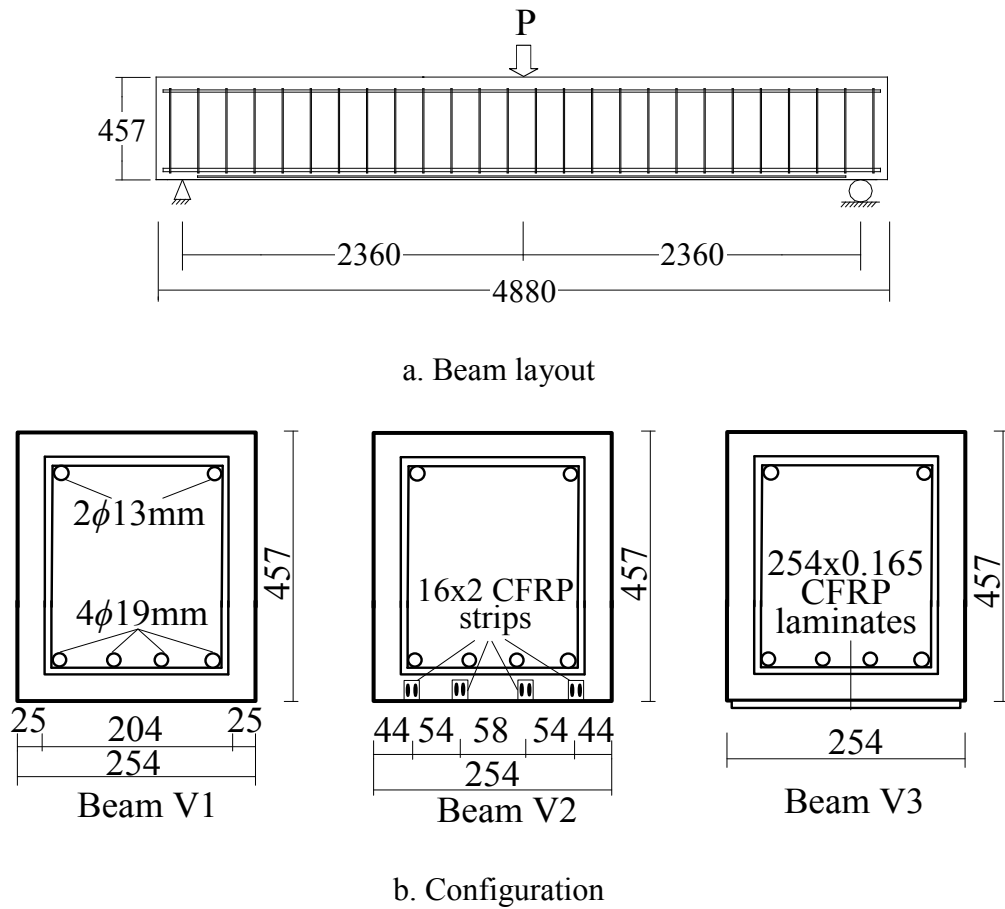


Figure 5.8 Configuration of tested beams for room temperature response validation (Units: mm)

Table 5.2 Configuration and properties of RC beams used for validation

Property		Beams V1, V2, V3	Beams V4, V5	Beams I, II, III & IV
Test condition		room temperature	ISO 834 standard fire	ASTM E119
Cross section (mm)		254×457	200×300	Flange: 432×127 Web: 228×279
Span (m)		4.72	3.0	3.66
Load/moment capacity ratio		monotonic	40%	I and II: 50% III and IV: 65%
f_c' (MPa)		34.5	40	53
Steel rebar	Tension	4 ϕ 19 mm	2 ϕ 16 mm	3 ϕ 19 mm
	Compression	2 ϕ 9 mm	2 ϕ 10 mm	4 ϕ 13 mm
	f_s (MPa)	576	550	455
FRP	FRP type (mm)	V2: Eight 16×2 CFRP strips V3: 254×0.165 CFRP laminates	V4: 2 ϕ 12 GFRP bars V5: 2 ϕ 9 CFRP bars	two 13.5×4.5 CFRP strips
	f_f (MPa)	V2: 2068 MPa V3: 643 MPa	GFRP: 1350 MPa CFRP: 1900 MPa	2510
Insulation type		None	V4: Promatect H V5: Aestuver	Tyfo CFP system

· f_c' , f_s , f_f is the compressive strength of concrete, strength of steel, and strength of FRP, respectively.

The three beams were analyzed under concentrated loading at mid-span, which was same as in tests. Based on the results from analysis, the moment capacities of Beam V1, Beam V2 and Beam V3, as predicted by the model, are 262, 391 and 400 kN-m, respectively, which are slightly lower than the measured values reported in tests (271, 399 and 410 kN-m). The slight difference between predicted and measured strength capacities is possibly due to the use of elasto-plastic stress-strain response for steel rebars, which may be lower than actual steel strength in the tested beams.

The predicted and measured load-deflection response for all three beams is compared in Figure 5.9. It can be seen that in the initial stages, all three beams exhibit a

linear response until cracking occurs in the beam (concrete). After that, the mid-span deflection increases at a higher rate due to decreasing stiffness resulting from cracking in the concrete beam. At this stage, the stresses in steel and FRP reinforcement increase at a faster rate until the steel rebars yield, and this can be seen in Figure 5.9 through the presence of another inflexion point on the load-deflection curve. In Beam V1, which is a conventional RC beam, the moment capacity reaches its peak once the steel rebar enters yielding plateau. However, in FRP strengthened beams (Beam V2 and Beam V3), the moment capacity keeps increasing with increased stress in FRP strips or laminates until the top concrete crushes.

In all three beams, the measured deflections at final stages are slightly higher than predicted values, and this is probably due to the fact that confinement effect on concrete is not considered in the model. Since Beam V2 and Beam V3 were designed to achieve same moment capacity from strengthening, their load-deflection response is quite similar under monotonic loading. It can be seen from Figure 5.9 that the overall load-deflection curves generated from the numerical model match well with measured data from tests. Thus, the above developed numerical model is deemed to be capable of predicting room temperature response of RC beams strengthened with NSM FRP reinforcement.

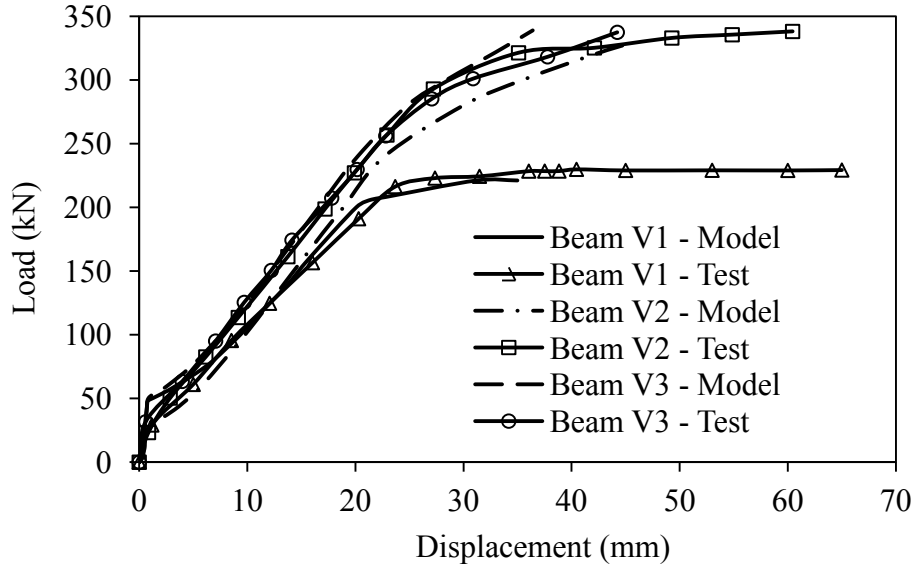


Figure 5.9 Load-deflection response in RC beams under monotonic loading (ambient condition)

5.4.2 Response under fire conditions - Rectangular beams

The validity of the above model in predicting fire response is established by comparing predictions from the analysis with fire test data on two beams (designated as Beam V4 and Beam V5) reported by Palmieri et al. (2012). The selected test beams are of 200×300 mm rectangular cross-section with an effective span of 3 m. The beam is divided into 40 segments, and the cross-section of each segment is subdivided into 700 elements of 10×10 mm. A finer mesh is adopted in the vicinity of NSM FRP and insulation area, as illustrated in Figure 5.1. The main flexural reinforcement in the beam comprise of two ϕ 16 mm steel rebars as primary reinforcement, and two ϕ 10 mm steel rebars as compression reinforcement. Two ϕ 12 GFRP rebars and two ϕ 9 CFRP rebars are installed as part of NSM strengthening in Beams V4 and V5, respectively. The high temperature properties of concrete and steel reinforcement follow the relations specified

in Eurocode 2 (2004). U-shaped fire protection boards are applied at the bottom surface and on two side surfaces of both beams. The insulation thickness at beam soffit of Beams V4 and V5 is 40 and 30 mm respectively and the insulation material is different from each other. Details of the beam configuration, and properties of concrete, steel and FRP, as reported from tests, are shown in Figure 5.10 and Table 5.2.

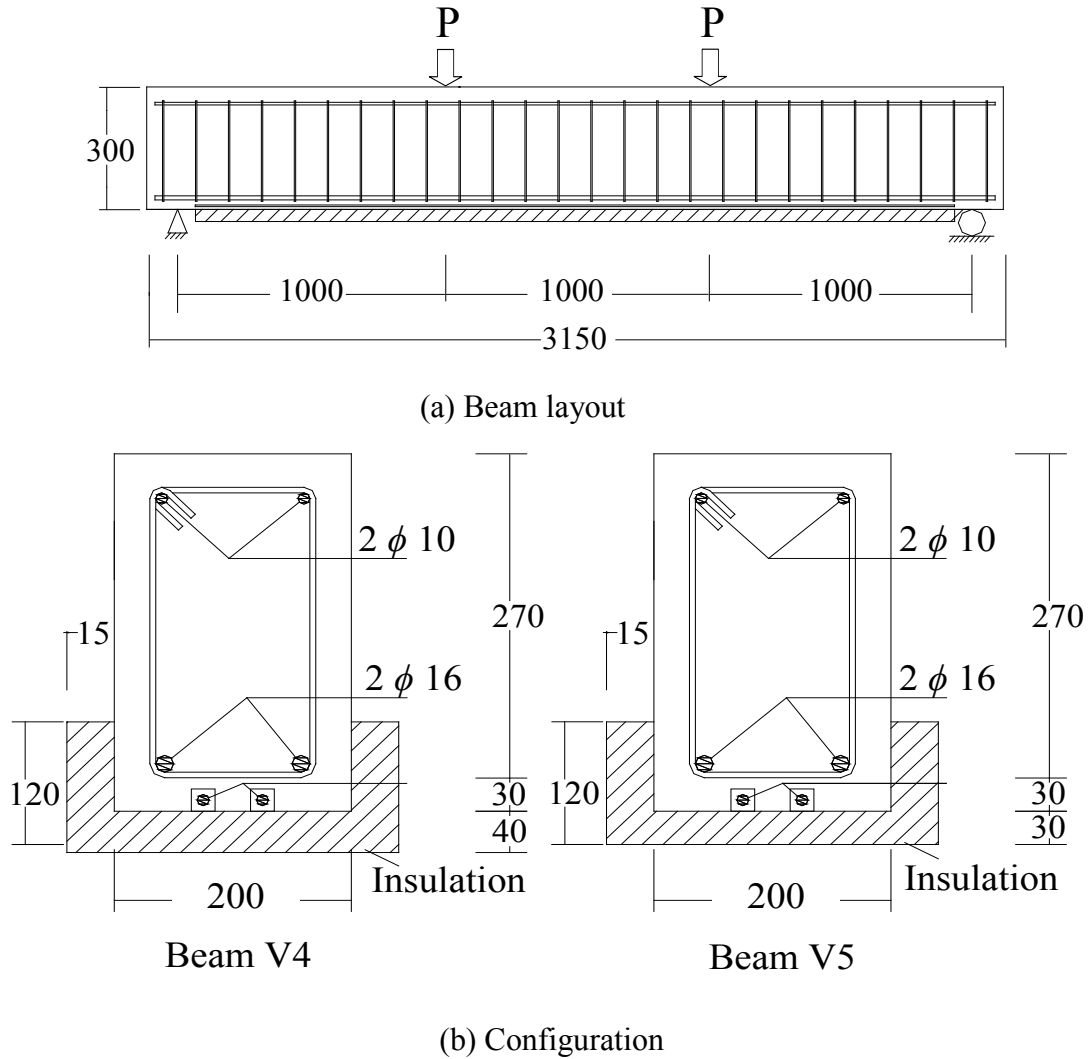


Figure 5.10 Configuration of tested beams for fire condition response validation (Units: mm)

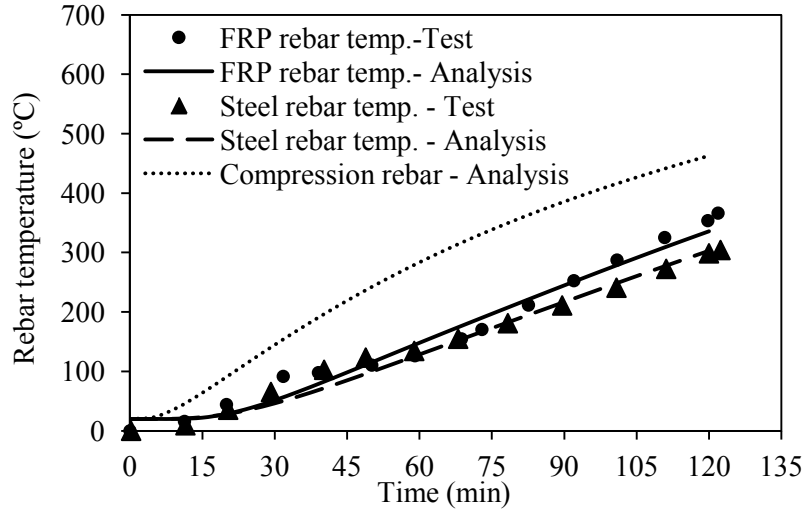
The validation process involved comparison of predicted thermal and structural response parameters from the analysis with measured values in fire tests. For thermal

response validation, the predicted FRP and steel rebar temperatures in Beam V4 are compared with measured temperatures in the fire test in Figure 11(a). It can be seen that the temperatures in both FRP and steel reinforcement remain low throughout the fire exposure duration, and this is mainly due to protection from fire insulation. At about 35 minutes, a plateau can be seen on the measured temperatures in FRP, and this is attributed to large heat consumption during the evaporation of free water in the surrounding concrete (when concrete temperature reaches about 100°C). However, numerical model does not predict this plateau at around 100°C, since the effect of moisture in concrete is not fully captured in high temperature thermal properties of concrete. The analysis predicted slightly unconservative temperatures as compared to measured values (in 60 to 90 minutes range), and this variation might be due to differences in high temperature thermal property relations of concrete and insulation used in the analysis and the actual properties of these materials used in the tested beam. The temperatures at other locations in concrete could not be compared since the authors did not report measured concrete temperatures in tests (Palmieri et al. 2012). However, a closer examination of predicted concrete temperatures from the model at various locations indicates the expected trend – lower temperatures at further distances from the fire exposed surface. Overall, the predicted temperature from analysis reasonably agrees with measured cross-sectional temperature in the test. Similar temperature comparison for Beam V5 indicates good agreement with measured temperatures.

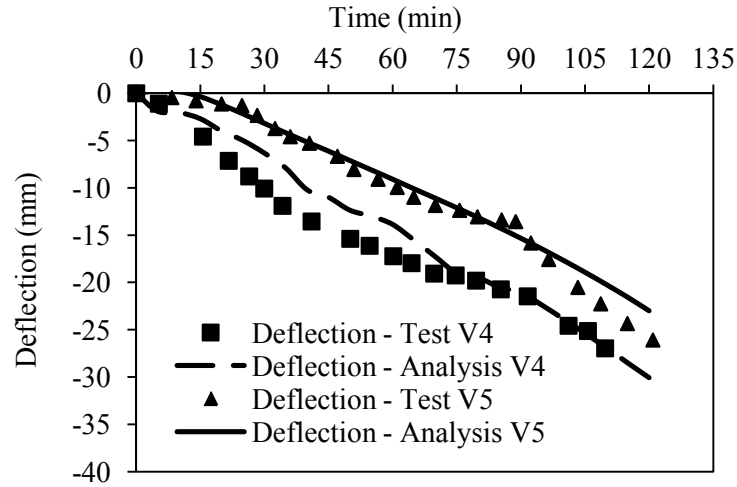
To validate structural response, the mid-span deflection predicted by the analysis is compared with measured deflections in fire tests for both beams (V4 and V5) in Figure 5.11 (b). It can be seen that both predicted and measured deflections increase at very slow

rate at the initial stage of fire exposure (up to 25 minutes), since FRP reinforcement has little reduction in its strength and stiffness, mainly due to lower temperature in FRP. With increase in fire exposure time, FRP and steel reinforcement experience relatively higher degradation in their strength and modulus, and thus the mid-span deflections in beams gradually increase as well. At about 90 minutes, the deflections in both beams increase at a higher rate and this might be due to yielding of steel (rebar).

Beam V4 experienced a relatively larger deflection than that of Beam V5 in the entire range of fire exposure, and this can be attributed to relatively lower stiffness and strength provided by GFRP reinforcement in Beam V4, as compared to CFRP reinforcement in Beam V5. Due to protection from U-shaped fire insulation, no FRP debonding observed in these two beams during the fire exposure time. Overall, the deflections predicted from the analysis agree reasonably well with the measured data from tests throughout the fire exposure time. Therefore, the macroscopic finite element model and the high-temperature constitutive relations in use are deemed appropriate for evaluating fire performance of RC beams strengthened with NSM FRP reinforcement.



(a) Rebar temperatures (Beam V4)



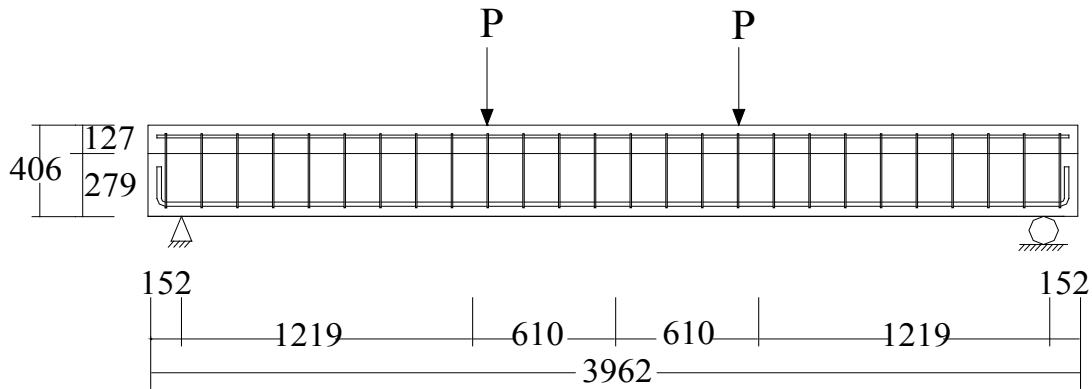
(b) Mid-span deflection (Beams V4 and V5)

Figure 5.11 Comparison of predicted and measured temperatures and mid-span deflections for Beams V4 and V5

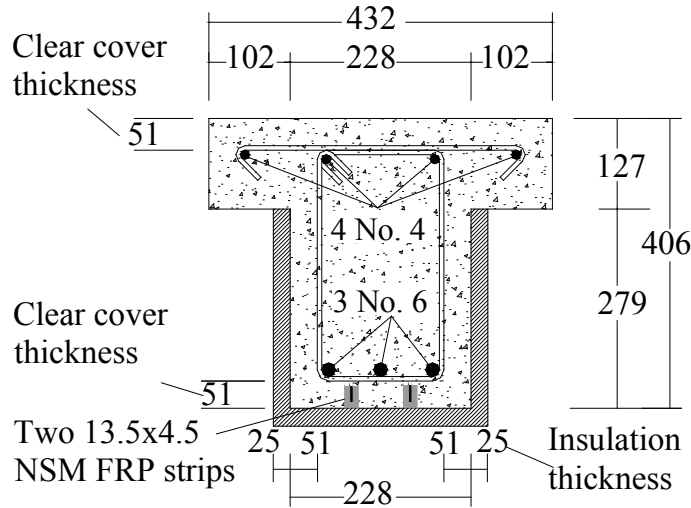
5.4.3 Response under fire conditions – T-beams

Further validation of the numerical model was undertaken by comparing response parameters against test data generated on NSM FRP strengthened T-beams as part of this

dissertation. Various thermal and structural response parameters, including cross-sectional temperatures, mid-span deflections, and axial restraint forces, were compared against the predictions from numerical model. In the analysis, the geometric and material properties of NSM FRP strengthened RC beams were taken to be as those given in the test program described in Chapter 4. Details on beam configuration and test conditions are shown in Figure 5.12 and Table 5.3.



(a) Beam layout

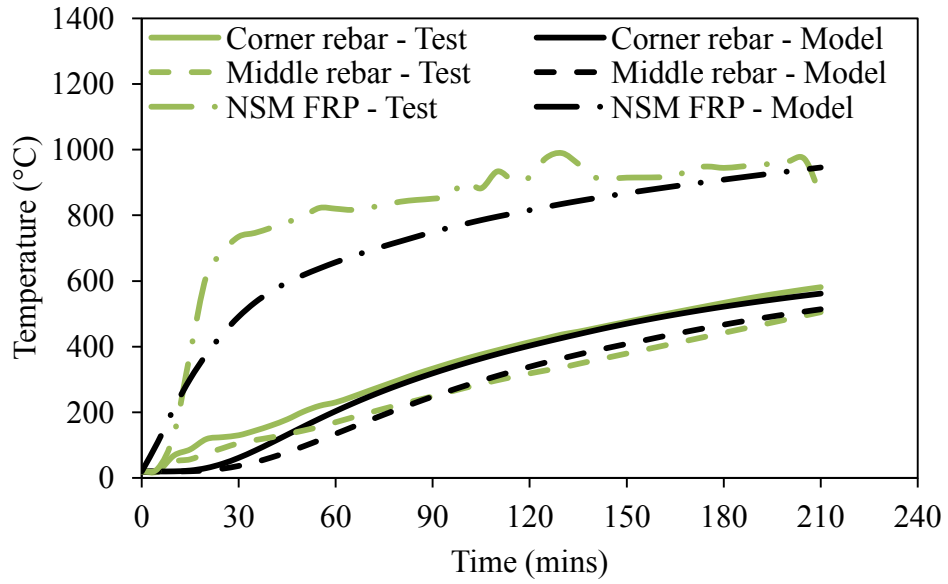


(b) Configuration

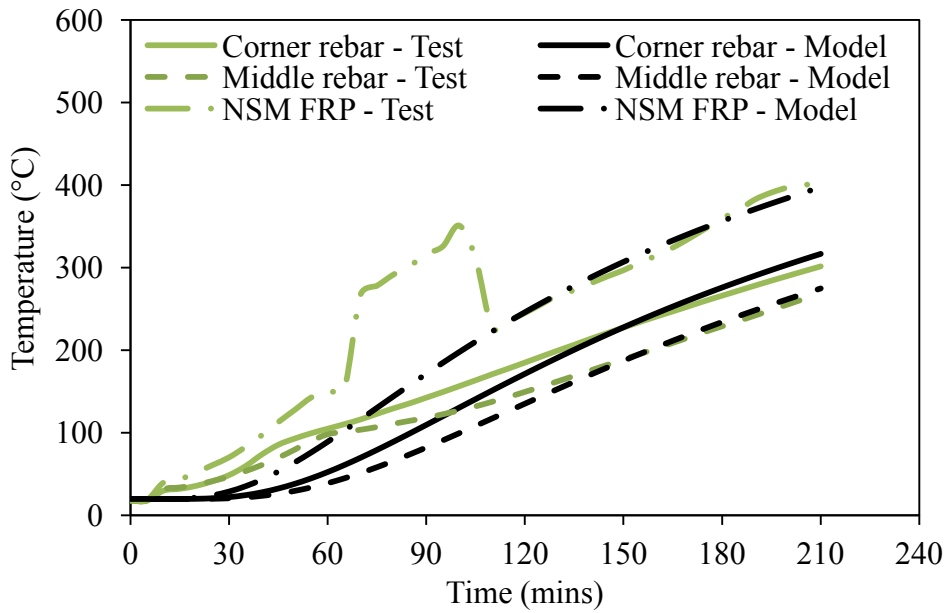
Figure 5.12 Configuration of tested T-beams for fire response validation (Units: mm)

To illustrate the usefulness of numerical model on tracing thermal response, predicted temperatures from numerical model are compared with measured temperatures at various locations, including NSM FRP, steel rebars, and concrete at different depth. Figure 5.13 plots a comparison of temperatures in steel rebar and NSM FRP for four tested beams. It can be seen that in uninsulated beam (Beam I), the temperature predictions in steel rebar match well with measured data throughout fire exposure duration. Temperature predictions in NSM FRP are lower than measured values. This is mainly attributed to the fact that burning of epoxy in Beam I leads to extremely high temperatures in NSM FRP in fire test, and heat transfer model does not account for effect of epoxy burning. However, since FRP lost most of strength beyond 800°C, this unconservative temperature prediction does not affect structural analysis of FRP strengthened RC beam.

In the case of insulated beams (Beams II to IV), there are some discrepancies between predicted and measured temperatures in steel rebar and NSM FRP, as shown in Figures 5.13(b)-(d). In fire tests, the measured cross-sectional temperatures in Beams III and IV are slightly higher than those in Beam II, due to large number of cracks that developed in these two beams resulting from higher load (stress) level. Numerical model does not account for the effect of crack formation and widening in insulation to thermal response, and thus gives identical temperature predictions in Beams II to IV, due to their same configuration and fire exposure. However, these temperature discrepancies are small, and temperatures in steel rebar and NSM FRP remain in a low range (below 400°C) throughout fire exposure duration. Thus this discrepancy does not significantly cause major different in predicted strength degradation in steel rebar or FRP.



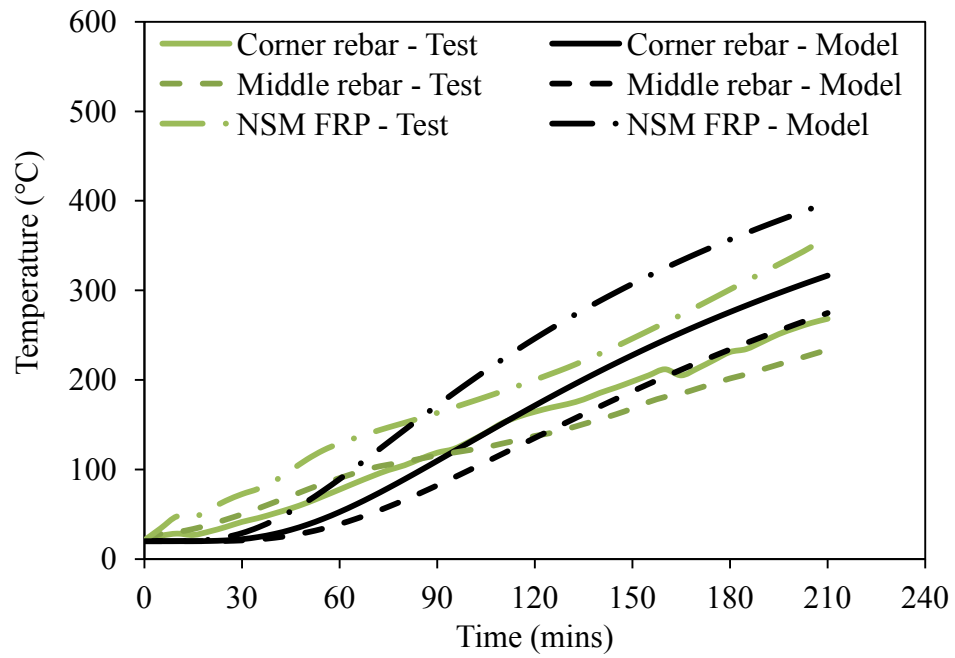
(a) Beam I



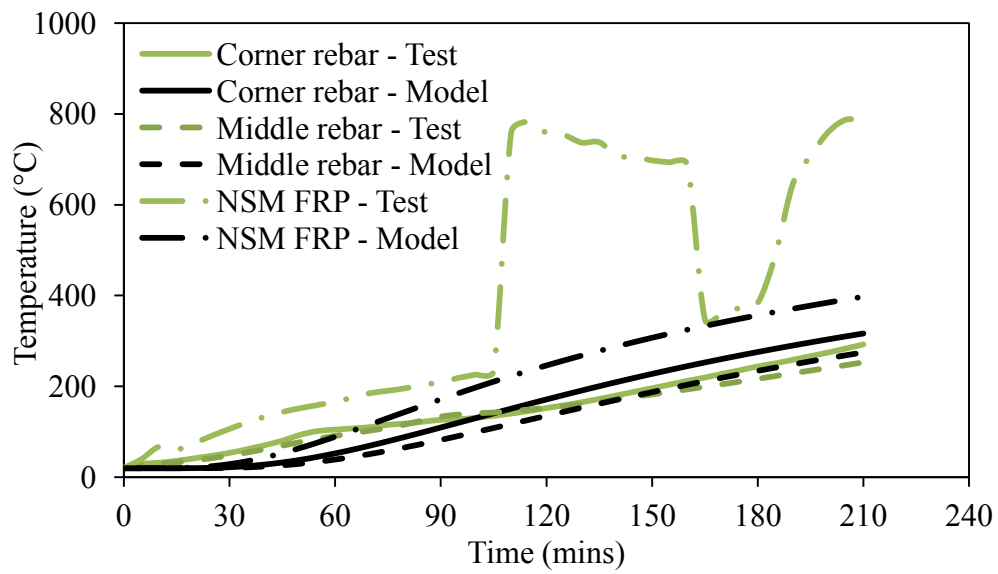
(b) Beam II

Figure 5.13 Comparison of predicted and measured temperatures in NSM FRP and steel rebar for MSU beams

Figure 5.13 (cont'd)

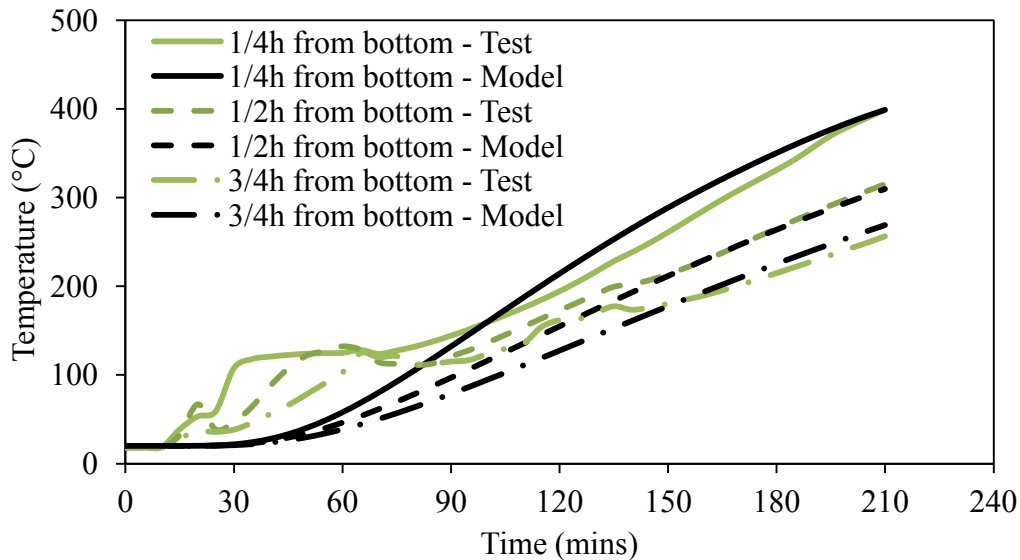


(c) Beam III



(d) Beam IV

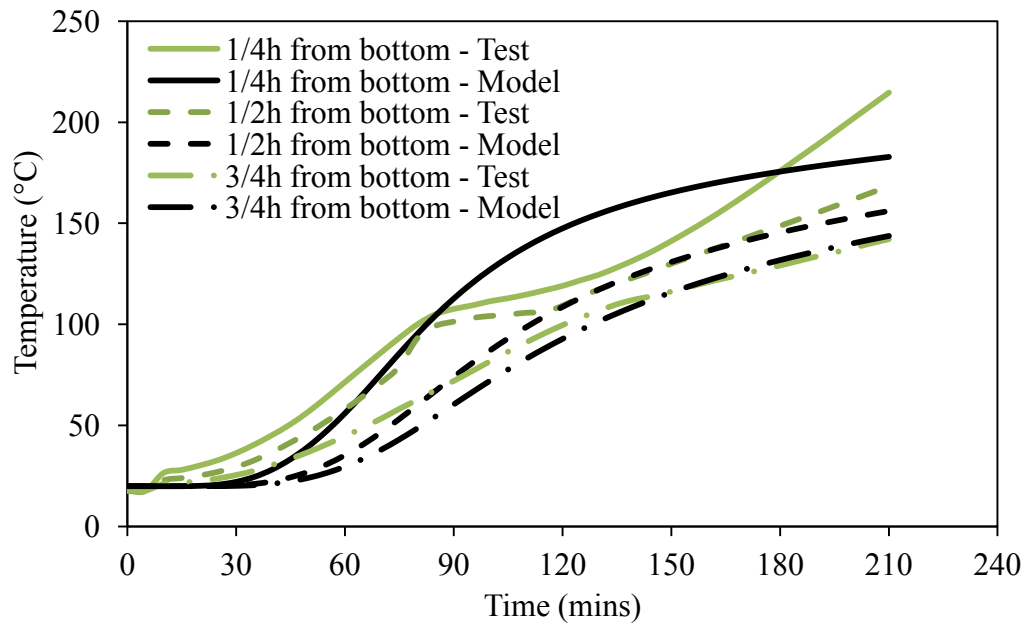
To further validate thermal response, the predicted and measured temperatures at various depth of concrete are compared in Figure 5.14. It can be seen in the early stage of fire exposure, the measured concrete temperatures usually increase at a faster rate than those from numerical analysis, especially for Beam I. This is mainly due to fast increase of fire temperature in the early stage. In the insulated beams (Beams II-IV), concrete temperatures remain in a low range (below 250°C) throughout fire exposure duration. This infers that there is not much strength degradation in concrete. A closer examination of plots shows that predicted and measured temperatures in concrete have some level of discrepancy for three insulated beams. This is mainly attributed to different levels of cracking in insulation and concrete resulting from different loading in these beams. Overall temperature predictions at various locations of beam cross section reasonably agree with temperature data obtained from fire tests.



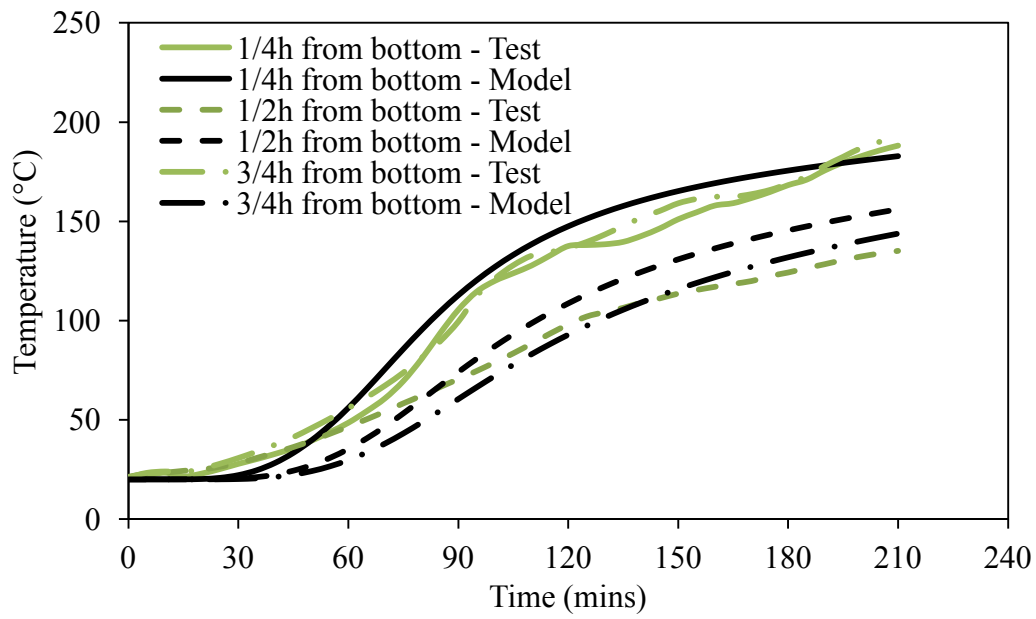
(a) Beam I

Figure 5.14 Comparison of predicted and measured temperatures in concrete for MSU beams

Figure 5.14 (cont'd)

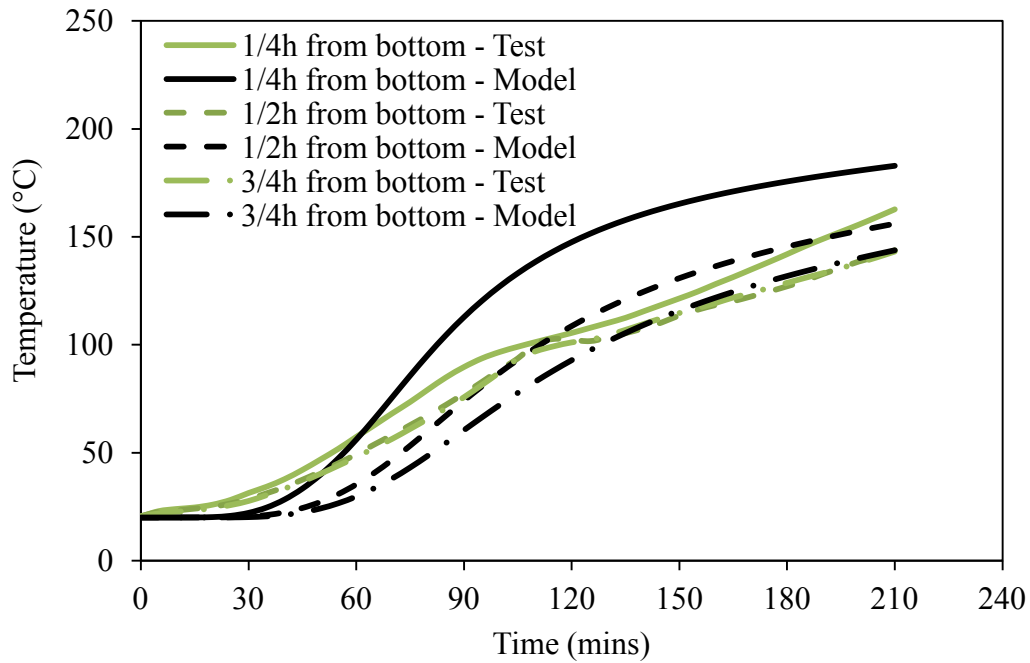


(b) Beam II



(c) Beam III

Figure 5.14 (cont'd)



(d) Beam IV

A comparison of predicted and measured mid-span deflections of four NSM FRP strengthened RC beams is plotted in Figure 5.15. In the case of uninsulated beam (Beam I), the mid-span deflection increased in a higher rate from the beginning of fire exposure. At 20 minutes, a small drop occurred in time-deflection curve of Beam I, which is probably caused by the slip of NSM FRP strips. The numerical model is capable of capturing this variation due to slip occurring in NSM FRP. Thereafter, the beam exhibited increasing deflections due to strength/stiffness degradation in steel rebar and FRP, however the beam did not fail for 210 minutes of fire exposure. Overall the deflection response obtained from numerical model matches measured data in Beam I in most fire exposure duration, and the predicted fire resistance also compares well with test data.

For the beams with fire insulation (Beams II-IV), the mid-span deflections remained low throughout fire exposure duration, especially for Beam II. This is mainly

due to lower temperatures in steel rebar, NSM FRP and in concrete. Beams III and IV were tested under a relatively higher loading (65%), and thus the deflections in these two beams increased at a faster rate during at later stage of fire exposure (after 160 minutes). Predicted deflections from numerical analysis are slightly smaller than those measured from tests, and this is probably because the numerical model does not account for the effect of crack widening in fire insulation. However, numerical model can properly simulate the axial restraint effect to structural behavior of the beam. It can be seen in Figure 5.15 that Beam III exhibits smaller deflections than those in Beam IV, and this is mainly due to the fire induced axial force which develops counter acting moment and thus reduces total moment on Beam III. Overall predicted deflections from the model have a good agreement with measured values throughout fire duration.

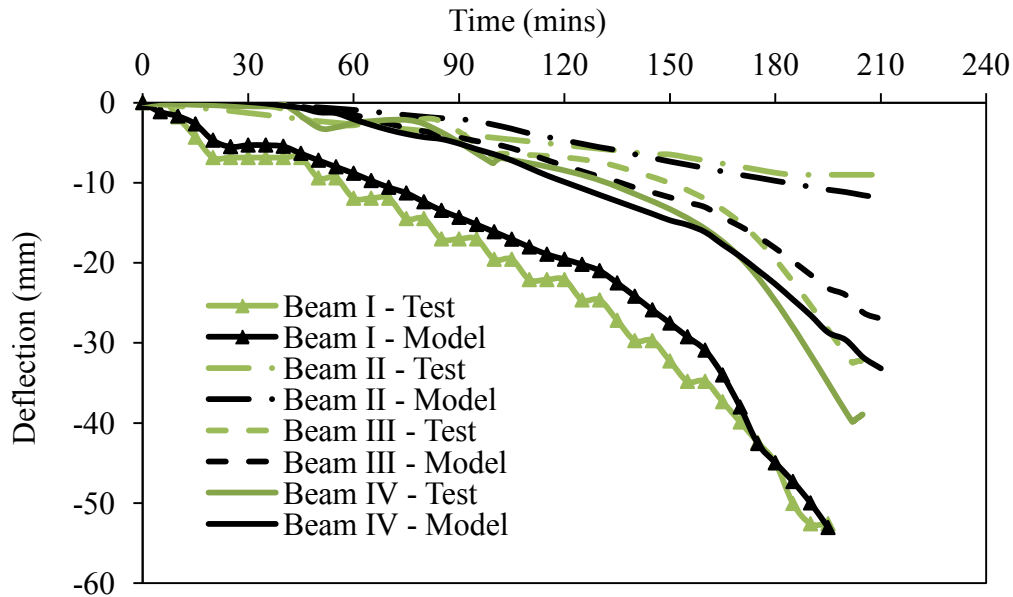


Figure 5.15 Comparison of predicted and measured mid-span deflection in T-beams

The predicted axial restraint force is also compared with measured force in the test, as shown in Figure 5.16. It can be seen that predicted axial restraint force reasonably

matches the data obtained from test. In the later stage of fire exposure (after 165 minutes), the measured axial force starts to decrease. This is probably due to faster increase in deflections after 165 minutes, and this might have caused slight detachment between concrete beam and restraint devices. However, in the numerical model, the predicted axial force keeps increasing due to thermal expansion of beam. Overall the axial restraint force and its influence on structural response of FRP strengthened beam are well traced by the numerical model.

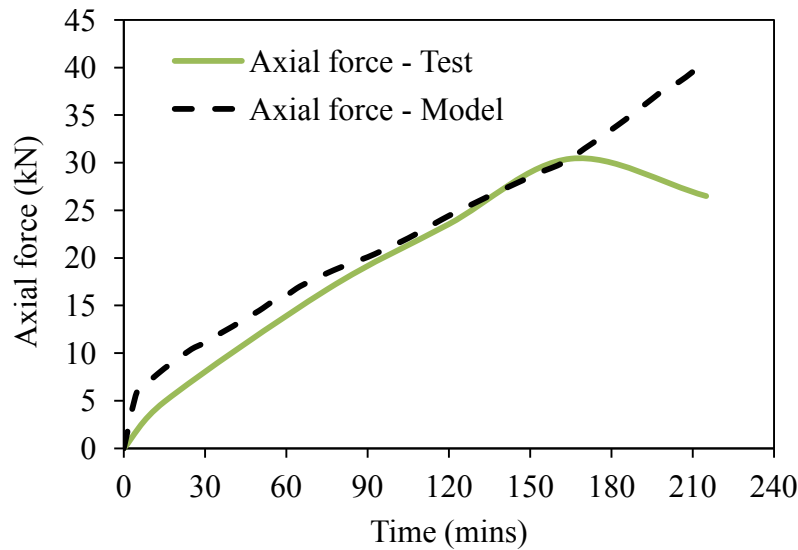


Figure 5.16 Comparison of predicted and measured axial forces in T- beams

5.5 Summary

In this chapter, a macroscopic finite element model available in literature is extended for evaluating fire response of NSM FRP strengthened RC beams. Fire resistance analysis in the model is conducted through three stages, namely, fire growth, thermal propagation and structural analysis. Various response parameters including cross sectional temperature, moment capacity and deflection, can be evaluated. The model

accounts for various cross sections, high temperature properties of constituent materials, as well as temperature induced bond-slip at NSM FRP/concrete interface.

The validity of this numerical model was established through comparing predictions from the model with test data on NSM FRP strengthened RC beam at both ambient and fire conditions. The comparison shows that predictions from numerical model have a good agreement with the measured values from fire resistance tests. This indicates that the developed macroscopic finite element model is capable of evaluating the fire response NSM FRP strengthened RC beams under various fire, loading, and boundary conditions. Therefore, this validated model can be further applied to undertake parametric studies to quantify the critical factors governing the fire resistance of NSM FRP strengthened RC beams.

CHAPTER 6

PARAMETRIC STUDIES

6.1 General

The fire response of FRP strengthened RC beams is influenced by a number of factors. Many of these influencing factors are interdependent and this makes fire resistance prediction quite complex. Thus the effect of these factors on fire response of FRP strengthened RC beam needs to be quantified through parametric studies. In this chapter, the numerical model validated in Chapter 5 is applied to evaluate the effect of various factors influencing fire response of NSM FRP strengthened RC beams and identify critical parameters. This is done through a set of parametric studies, wherein the response of NSM FRP strengthened RC beam is evaluated by varying each parameter over a wide range. The results from parametric studies can be utilized to develop design guidelines on fire response of RC beams strengthened with NSM FRP reinforcement. Details on procedure and results of parametric studies are discussed in the following sections.

6.2 Critical Factors Influencing Fire Resistance

Critical factors influencing fire resistance of RC beams with external FRP laminates or internal FRP rebars have been evaluated in previous studies (Ahmed and Kodur 2010, Yu and Kodur 2013). However, there are no studies in the literature to quantify the factors influencing on fire response of NSM FRP strengthened RC beams.

Previous studies on fire response of FRP strengthened RC beams indicate that temperature-induced FRP degradation is a primary factor causing the failure of strengthened beams under fire conditions. Thus the influence of FRP strengthening type, NSM FRP locations, and reinforcement ratio of FRP to steel needs to be gauged. Further, concrete strength, external loading and boundary conditions, and fire exposure have critical influence on fire response, and they are also main factors to be investigated. In addition, fire insulation can significantly enhance the behavior of FRP strengthened RC beam under fire conditions. However, the thickness and layout of insulation have to be optimized for performance and economy. Therefore, an optimal insulation scheme is also developed for NSM FRP strengthened RC beams.

6.3 Parametric Studies

The influence of various parameters to fire response is evaluated through numerical analysis on a typical NSM FRP strengthened RC beam in buildings. The configuration, material properties, and boundary conditions of this beam are varied over a wide range and thus the influence of each parameter to fire response can be evaluated. Specific parameters in the study include FRP strengthening type, NSM FRP location, reinforcement ratio of NSM FRP and steel, load level, axial restraint stiffness, concrete strength, fire scenario, insulation layout. The details on the influence of each parameter are presented in the following sections.

6.3.1 Beam configuration and parameters in study

A simply supported RC beam with NSM FRP, designated as Beam A, was selected as the primary beam for parametric studies. This beam is of 254×457 mm

rectangular cross-section with a span length of 4.72 m. The primary reinforcement is composed of four ϕ 19 mm steel rebars, while the compression reinforcement is two ϕ 12.7 mm steel rebars. Four 16×4 mm CFRP strips are used as NSM strengthening at tension face of the beam. The beam is subjected to a uniform distributed load, which is equivalent to 50% of the room temperature capacity of beam. In the analysis, the configuration parameter and boundary conditions of this o beam were varied in a wide range, and thus a number of beams were generated for parametric studies. The detailed information on geometric and material properties is shown in Table 6.1 and Figure 6.1.

Table 6.1 Geometric and material properties of FRP strengthened RC beams
used in parametric study

Property		Values
Cross-section (mm)		254×457
Span (m)		4.72
Loading	Magnitude	70 kN/m, 50% load ratio
	Load type	Uniformly distributed load
Concrete	Aggregate	Carbonate
	f_c' (MPa)	34.5
	Cover thickness	25 mm
Steel rebar	Top rebar	4 ϕ 19
	Bottom rebar	2 ϕ 12.7
	f_y (MPa)	576
NSM CFRP	Dimension	Four 16 × 4 mm strips
	f_{FRP} (MPa)	2068
	E_{FRP} (GPa)	131
	Groove size	25 × 10 mm
External CFRP	Dimension	254 × 0.165 mm laminates
	f_{FRP} (MPa)	634
	E_{FRP} (GPa)	46.4

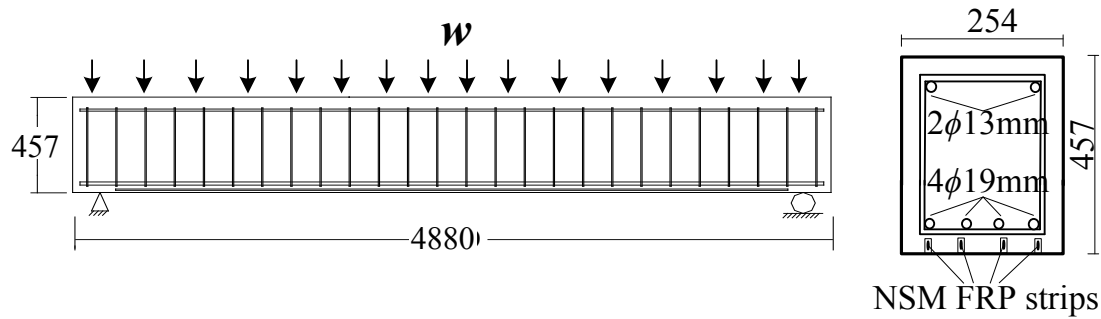


Figure 6.1 Configuration and elevation of NSM FRP strengthened RC beam (Beam A) for parametric study (Units: mm)

To quantify the influence of various factors to fire resistance of RC beams strengthened with NSM FRP, eight groups of analysis were carried out using the numerical model validated in Chapter 5. In each group of analysis, one parameter was varied within a practical range, while all other properties were kept constant.

- In Group 1 beams, the type of FRP strengthening was varied, and the fire response of an RC beam under different configurations, namely, unstrengthened, strengthened with externally bonded FRP, and strengthened with NSM FRP, was compared under the same loading level. The amount of FRP strengthening was selected such that the same level of moment capacity is achieved in different strengthening arrangement.
- In Group 2 beams, the location of NSM FRP was varied from corner of beam soffit to middle, to demonstrate the influence of NSM reinforcement location on fire response of the strengthened beams.
- In Group 3 beams, the area of FRP strips or the strength of steel rebar was varied, and thus the reinforcement ratio of steel rebar and FRP strips is different in each beam. The purpose of this analysis is to quantify the influence of the

reinforcement ratio of FRP to steel on fire resistance of NSM FRP strengthened RC beams.

- In Group 4, the compressive strength of concrete was varied from 35 to 60 MPa, to study the influence of concrete strength to fire response of NSM FRP strengthened beams.
- In Group 5, the load level on the strengthened beam was varied from 40-70% of room temperature capacity, to quantify the influence of load level on fire resistance of NSM FRP strengthened beams.
- In the analysis of Group 6 beams, axial restraint at the supports of beam was varied from zero to 200 kN/mm, to evaluate the influence of axial restraint on fire response.
- In Group 7 beams, the beams were exposed to four different fire scenarios to study the response of NSM FRP strengthened RC beams under different fire scenarios.
- Finally in Group 8, NSM FRP strengthened RC beam (Beam A) was provided with fire insulation, and the insulation thickness and its geometric layout were varied to develop an optimum insulation scheme.

Detailed range for each parameter is tabulated in Table 6.2. The primary beam, Beam A, in each group is also marked in the table.

6.3.2 Material properties

The range of properties of constituent materials used for parametric studies is consistent with those used in field applications. At room temperature, the compressive

strength of concrete (f'_c) is 34.5 MPa, and the yield strength of steel (f_y) is 576 MPa. The high temperature properties of concrete and steel rebar are assumed to vary as per ASCE relations (Lie 1992). CFRP laminate (used in Beam I-2 as shown in Table 6.2) has a strength of 634 MPa, and elastic modulus of 46.4 GPa; while CFRP strips used in NSM strengthened beams has a strength of 2068 MPa, and elastic modulus of 131 GPa. The high temperature tensile strength properties and bond strength properties of NSM CFRP follow the empirical relations proposed in Chapter 3, and the variations of those properties for external FRP laminates follow the empirical relations reported by Bisby et al. (2005). For VG (Vermiculite-Gypsum) fire insulation, the temperature-dependant thermal capacity and thermal conductivity are assumed to follow the properties reported in the literature (Bisby 2003).

The detailed material properties for concrete, steel rebar, and NSM FRP are tabulated in Table 6.1. Temperature-dependant property relations are provided in Appendix A.

Table 6.2 Critical factors investigated in parametric study

Factor	Designation	Variable
FRP strengthening	Beam I-1	RC beam
	Beam I-2	External CFRP strengthened RC beam
	Beam I-3	NSM CFRP strengthened RC beam (Beam A)
NSM FRP location	Beam II-1	Uniformly distributed at beam soffit (Beam A)
	Beam II-2	Close to corner at beam soffit
	Beam II-3	At middle of beam soffit
Reinforcement ratio of FRP to steel	Beam III-1	4 ϕ 19 rebar and 4 16 $\times$ 4mm FRP strips ($f_y = 576$ MPa, $f_f = 2076$ MPa) (Beam A)
	Beam III-2	4 ϕ 19 rebar and 4 16 $\times$ 4mm FRP strips ($f_y = 461$ MPa, $f_f = 2076$ MPa)
	Beam III-3	4 ϕ 19 rebar and 4 16 $\times$ 2mm FRP strips ($f_y = 576$ MPa, $f_f = 2076$ MPa)
Concrete strength	Beam IV-1	$f_c' = 34.5$ MPa (Beam A)
	Beam IV-2	$f_c' = 50$ MPa
	Beam IV-3	$f_c' = 60$ MPa
Load level	Beam V-1	Load is equivalent to 40% of room temperature capacity
	Beam V-2	Load is equivalent to 50% of room temperature capacity (Beam A)
	Beam V-3	Load is equivalent to 60% of room temperature capacity
	Beam V-4	Load is equivalent to 70% of room temperature capacity
Axial restraint	Beam VI-1	$k = 0$ (Beam A)
	Beam VI-2	$k = 5$ kN/mm
	Beam VI-3	$k = 50$ kN/mm
	Beam VI-4	$k = 100$ kN/mm
	Beam VI-5	$k = 200$ kN/mm
Fire scenario	Beam VII-1	ASTM E119 (Beam A)
	Beam VII-2	ASTM Hydrocarbon fire
	Beam VII-3	Design fire I
	Beam VII-4	Design fire II
Insulation layout	Beam VIII-1	No insulation (Beam A)
	Beam VIII-2	Insulation thickness = 15 mm, depth = 75 mm
	Beam VIII-3	Insulation thickness = 25 mm, depth = 75 mm
	Beam VIII-4	Insulation thickness = 35 mm, depth = 75 mm
	Beam VIII-5	Insulation thickness = 25 mm, depth = 50 mm
	Beam VIII-6	Insulation thickness = 25 mm, depth = 75 mm
	Beam VIII-7	Insulation thickness = 25 mm, depth = 100 mm

6.3.3 Discretization and analysis details

For numerical analysis, the beam is discretized into 40 segments along its length, with smaller segments at critical regions moment such as mid-span and a smaller length of segment is divided around mid-span, as depicted in Figure 6.2. The central section of the segment, which is assumed to represent the behavior of each segment, is further divided into number of elements. A finer mesh is applied in critical zones (steel rebar, NSM FRP, and insulation) along the beam cross section to achieve better accuracy in the numerical analysis. All beams have simply supported conditions and are loaded with uniformly distributed load along the span. This applied load is equivalent to 50% of the room temperature capacity of each beam, which is calculated as per ACI 318 (2011) and ACI 440.2 (2008) provisions. The beams are exposed to ASTM E119 standard fire for four hours till failure is attained. Cross-sectional temperatures, moment capacity and deflections are output at each time step (five minutes) to evaluate fire response of FRP strengthened RC beams.

6.3.4 Failure criteria

Based on the discussion in Chapter 5, three limiting criteria are applied to evaluate the fire resistance of NSM FRP strengthened RC beam, namely moment capacity, mid-span deflection, and rate of deflection. Steel rebar temperature limit is not considered as one failure criteria, since NSM FRP might contribute to strength capacity of the beam to some extent even at later stage of fire exposure. Also, this criteria is not sensitive to the failure caused by load level and boundary conditions. Partial debonding of NSM FRP or external FRP laminate does not define the failure of FRP strengthened

beams, since both literature review and experimental studies indicate the strengthened beams still possess good load resistance after FRP debonding. Overall, the strength capacity, deflection, and rate of deflection are the most realistic criteria for evaluating the failure of NSM FRP strengthened beams under fire conditions.

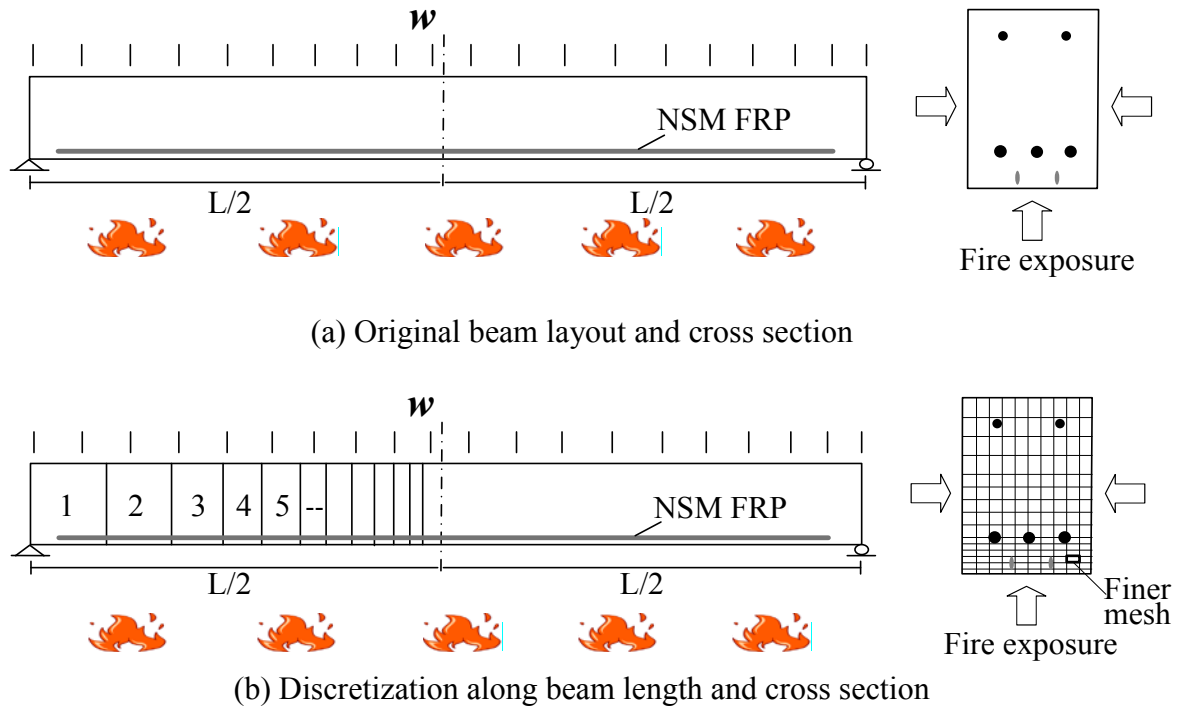


Figure 6.2 Layout of NSM FRP strengthened RC beam and discretization along beam length and cross section

6.4 Results of Parametric Studies

Results from fire resistance analyses are utilized to gauge the influence of critical factors on fire response of NSM FRP strengthened RC beams. Various output parameters, including temperature in steel rebar and FRP, deflection, moment capacity, and fire resistance, are generated to quantify the influence of each factor. Table 6.3 provides a comparison of results from parametric studies. Detailed analysis results for each factor are discussed in the following sections.

Table 6.3 Summary of fire resistance values for the beams in parametric studies

Parameter	Beam designation	Ratio of FRP to steel ¹	LR ² (%)	Fire scenario	k ³ (kN/mm)	Insulation	FR ⁴ (min)
FRP strengthening	Beam I-1	0.81	50	ASTM E119	0	No	160
	Beam I-2	0.81	50	ASTM E119	0	No	90
	Beam I-3	0.81	50	ASTM E119	0	No	110
NSM FRP location	Beam II-1	0.81	50	ASTM E119	0	No	110
	Beam II-2	0.81	50	ASTM E119	0	No	105
	Beam II-3	0.81	50	ASTM E119	0	No	115
Ratio of FRP to steel	Beam III-1	0.81	50	ASTM E119	0	No	110
	Beam III-2	1.01	50	ASTM E119	0	No	80
	Beam III-3	0.41	50	ASTM E119	0	No	135
Concrete strength	Beam IV-1	0.81	50	ASTM E119	0	No	110
	Beam IV-2	0.81	50	ASTM E119	0	No	115
	Beam IV-3	0.81	50	ASTM E119	0	No	120
Load level	Beam V-1	0.81	40	ASTM E119	0	No	140
	Beam V-2	0.81	50	ASTM E119	0	No	110
	Beam V-3	0.81	60	ASTM E119	0	No	95
	Beam V-4	0.81	70	ASTM E119	0	No	65
Axial restraint	Beam VI-1	0.81	50	ASTM E119	0	No	110
	Beam VI-2	0.81	50	ASTM E119	5	No	115
	Beam VI-3	0.81	50	ASTM E119	50	No	125
	Beam VI-4	0.81	50	ASTM E119	100	No	135
	Beam VI-5	0.81	50	ASTM E119	200	No	140
Fire scenario	Beam VII-1	0.81	50	ASTM E119	0	No	110
	Beam VII-2	0.81	50	Hydrocarbon	0	No	75
	Beam VII-3	0.81	50	Design I	0	No	80
	Beam VII-4	0.81	50	Design II	0	No	155
Insulation thickness (mm)	Beam VIII-1	0.81	50	ASTM E119	0	No	110
	Beam VIII-2	0.81	50	ASTM E119	0	15	190
	Beam VIII-3	0.81	50	ASTM E119	0	25	235
	Beam VIII-4	0.81	50	ASTM E119	0	35	255
Insulation depth (mm)	Beam VIII-5	0.81	50	ASTM E119	0	50	200
	Beam VIII-6	0.81	50	ASTM E119	0	75	235
	Beam VIII-7	0.81	50	ASTM E119	0	100	250

1. Ratio of FRP to steel is the ratio of product of cross sectional area and tensile strength of FRP to that of steel.

2. LR means load ratio.

3. Axial restraint stiffness

4. Fire resistance

6.4.1 Effect of FRP strengthening

Three beams, designated as Beam I-1, Beam I-2 and Beam I-3, were analyzed for evaluating comparative fire response of RC beam with different FRP strengthening. Beam I-1 is a conventional RC beam with four ϕ 19 mm steel rebars as tensile reinforcement, while Beams I-2 and I-3 are Beam I-1 strengthened with externally bonded (EB) FRP laminates and NSM FRP strips respectively. Beams I-2 and I-3 are designed to yield similar flexural capacity and based on the room-temperature analysis, the nominal flexural capacities of Beams I-1, I-2 and I-3 are 246, 392, and 393 kN·m, respectively. Elevation and configuration of three beams are shown in Figure 6.3.

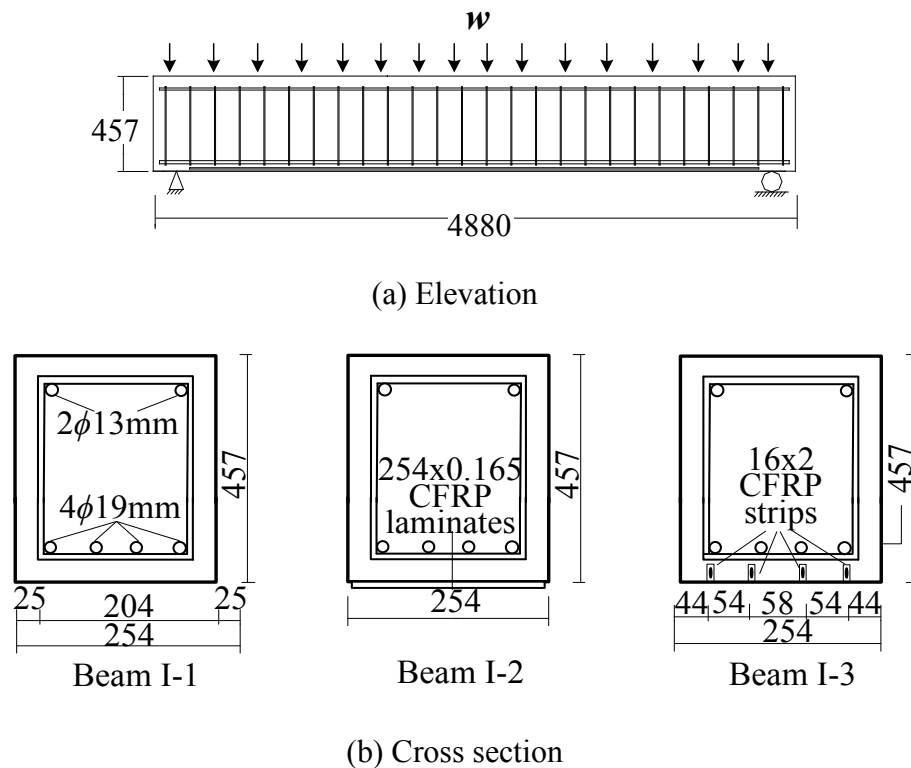


Figure 6.3 RC beams analyzed for studying the effect of FRP strengthening (Unit: mm)

The thermal response of three beams (Beams I-1, I-2 and I-3) is compared in Figure 6.4, by plotting steel and FRP reinforcement temperatures as a function of fire

exposure time. The temperature at the center of steel rebar and FRP strip and the average temperature of FRP laminate are selected for comparison. It can be seen that the steel rebar temperatures in Beams I-1 and I-3 are identical, since the rebars are located at the same location and the concrete cover depth is identical. However, the temperature rise in steel rebar in Beam I-2 is slower than those in Beams I-1 and I-3. This can be attributed to the fact that FRP laminates slow down the heat transfer from fire zone to the bottom surface of the beam due to relatively lower thermal conductivity of FRP laminate.

Temperatures in FRP reinforcement are significantly different in Beams I-2 and I-3 during the entire fire exposure time, as plotted in Figure 6.4. In Beam I-2 (with externally strengthened FRP), the temperature of FRP laminate increases at a faster rate from the start of fire exposure, and reaches the critical temperature of CFRP (around 350°C) in 13 minutes. This is due to direct exposure of FRP laminate to fire. In Beam I-3 (NSM FRP strengthened RC beam), the temperature rise in NSM FRP strip is at a much lower rate than that of FRP laminate in Beam I-2, and this can be attributed to thermal protection effect provided through concrete cover and near surface adhesive on FRP. NSM FRP in Beam I-3 also reaches critical temperature in about 30 minutes of fire exposure, and NSM strips retain more strength throughout fire exposure duration. Therefore, the type of strengthening (external or NSM) influences the rate of temperature rise in steel and FRP reinforcement under fire conditions.

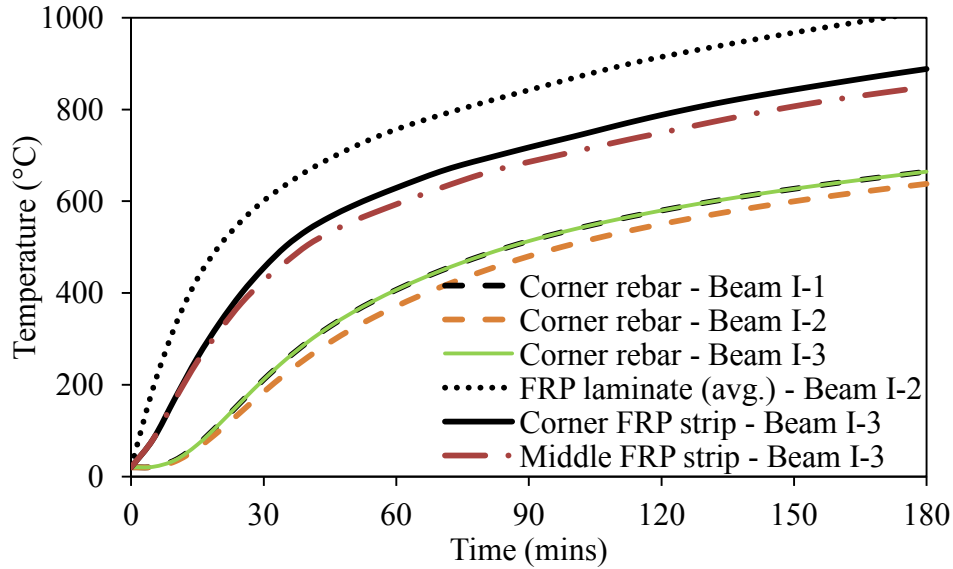


Figure 6.4 Effect of FRP strengthening type on temperature rise in steel rebar and FRP

To illustrate the comparative structural response of these three beams with different strengthening system, the variation of moment capacity with fire exposure time is plotted in Figure 6.5. For the control beam (Beam I-1), there is no drop in the moment capacity till about 55 minutes, and this is mainly due to the fact that there was no strength loss in steel rebars since the temperature in rebar stays quite low (below 350°C). At about 60 minutes, the temperature in corner steel rebar increases to about 400°C and consequently steel rebars start to lose some of their strength and elastic modulus properties. Thereafter, the moment capacity in Beam I-1 decreases gradually till failure occurs. For the RC beam with externally bonded FRP laminate (Beam I-2), the moment capacity decreases drastically in initial stages of fire exposure, and this can be attributed to significant drop in the strength of FRP laminates due to a faster rise in FRP temperature, which reaches about 500°C at 15 minutes. At about 20 minutes, the contribution of FRP to the strength of the beam ceases, and then this beam behaves similar to that of control RC beam. As to NSM strengthened beam, Beams I-3 retains

much higher moment capacity than that of Beam I-2 during the entire fire exposure time. The main reason for this higher moment capacity can be attributed to lower loss of strength in NSM strips, which is due to slower temperature rise resulting from concrete cover to NSM FRP strips. Also, the bond between NSM FRP strips and concrete remains effective for a longer duration and thus NSM FRP strips continue to contribute to some level of moment capacity till about two hours. Therefore, NSM strengthened beam achieves higher fire resistance than that of externally strengthened beam.

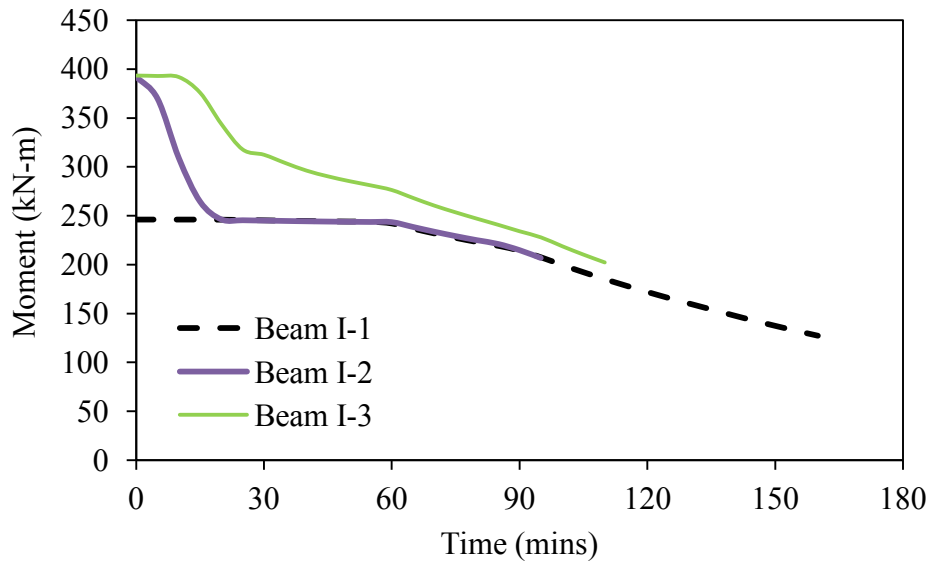


Figure 6.5 Effect of FRP strengthening type on variation of moment capacity of RC beams

To further illustrate the variation in structural response, mid-span deflection in three beams is plotted as a function of fire exposure time in Figure 6.6. At the initial stages of fire exposure, the resulting deflections in all three beams are similar, due to the same level of loading and similar fire exposure conditions. After about 30 minutes, Beams I-2 and I-3 experience deflections at an accelerated rate as compared to the control beam (Beam I-1). This can be attributed to deteriorating stiffness in Beams I-2 and I-3

that results from decreasing elastic modulus in FRP reinforcement at elevated temperature. After about 90 minutes into fire exposure, temperature in inner steel rebars reaches about 400°C and this induces higher degradation in strength and elastic modulus of steel rebars (400°C). This in turn results in significant rise in deflection in all three beams until failure occurs. Overall, at the same load level (applied moment to capacity ratio), the conventional RC beam (Beam I-1) exhibits the highest fire resistance (160 minutes), while the RC beam with externally bonded FRP (Beam I-2) exhibits the lowest fire resistance of 95 minutes. The beam strengthened with NSM FRP strips (Beam I-3) has a fire resistance of 110 minutes. Of the two FRP strengthened beams, NSM FRP strengthened beam exhibits better structural response than the one with externally bonded FRP.

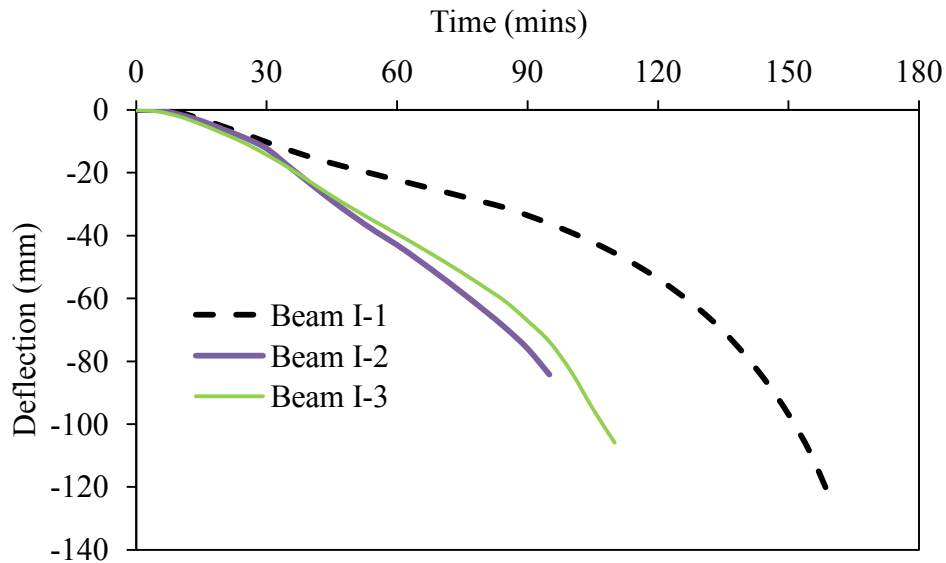


Figure 6.6 Effect of FRP strengthening type on the variation of mid-span deflection of RC beams

6.4.2 Effect of NSM FRP location

The strength degradation in NSM FRP reinforcement plays a critical role on fire resistance of strengthened beams as seen from above analysis. Thus, the location of NSM FRP rods or strips within beam cross-section is an important factor governing fire response, since temperature rise in NSM FRP strip/rod depends on their locations. Three NSM FRP strengthened RC beams, designated as Beam II-1, II-2, and II-3, were studied as part of this parametric study. In Beam II-1, four FRP strips are evenly distributed at the beam soffit. Beams II-2 and II-3 have the same amount of NSM FRP strips, but NSM FRP strips are located around corner area at beam soffit in Beam II-2, and around middle area at beam soffit in Beam II-3 respectively, as shown in Figure 6.7. Results from fire resistance analysis on Beams II-1 to II-3 are used to illustrate the comparative fire response of RC beams with NSM FRP at various locations.

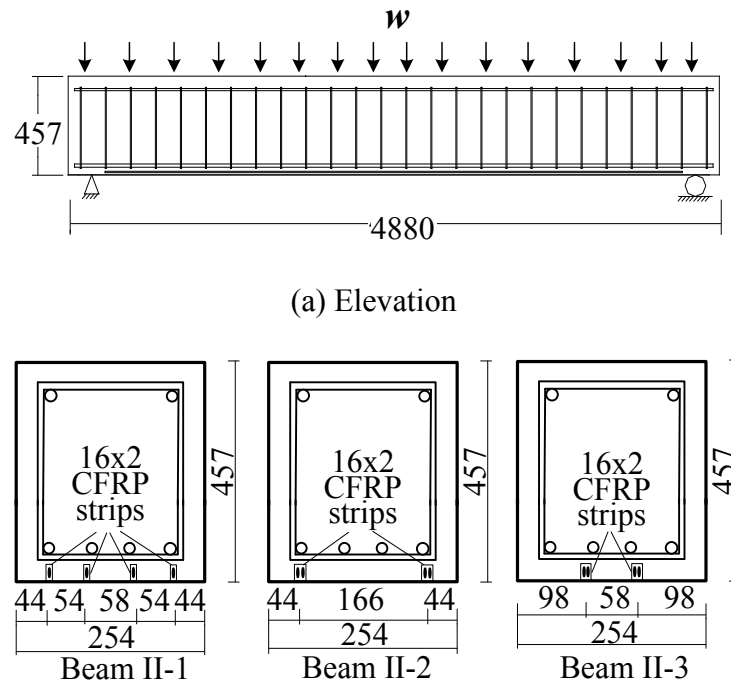


Figure 6.7 RC beams analyzed for studying the effect of NSM FRP location (Units: mm)

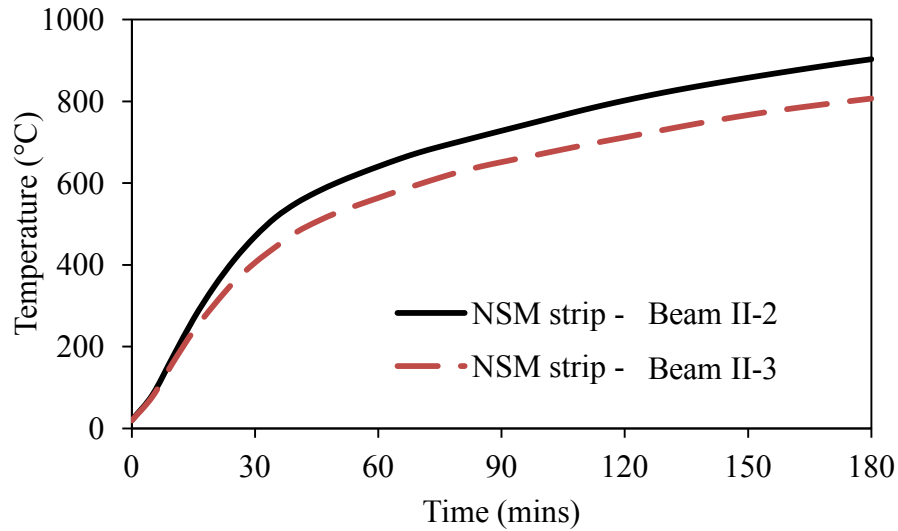


Figure 6.8 Effect of FRP location on temperatures rise in FRP

The variation of temperatures in NSM FRP strips as a function of fire exposure time is plotted in Figure 6.8. It can be seen the temperatures in NSM FRP are different from one beam to another, and they are highly dependent on the location of FRP strips. In Beam II-3, the temperature in NSM FRP strips increases at a slower rate than that in Beam II-2, and this is mainly due to the location of FRP strip at middle of the beam soffit, which is farther from fire-exposed side surfaces. The slower temperature rise in NSM FRP strips in Beam II-3 helps to retain strength during fire exposure, and this leads to higher fire resistance of NSM FRP strengthened beams. Therefore, the location of NSM FRP affects fire response of NSM FRP strengthened beams. The temperature rise in steel rebar is identical in all three beams, since the location of steel rebar remains the same.

The structural response of Beams II-1, II-2 and II-3 is shown in Figure 6.9, by plotting the variation of their moment capacity as a function of fire exposure time. It can be seen in Figure 6.9, the degradation of moment capacity in three beams is similar, and the slight difference among these beams is attributed to different level of contribution

from NSM FRP strips. The beam with FRP strips at the center of beam soffit (Beam II-3) is able to retain more strength in FRP, and thus attains higher moment capacity. This difference in moment capacity produces 5-10 minutes variation of fire resistance. Beam II-1 achieves fire resistance of 110 minutes, while Beams II-2 and II-3 yield fire resistance of 105 and 115 minutes, respectively. Thus, besides satisfying the basic requirement of spacing and groove size the location of NSM FRP strips should be placed close to mid-portion of beam soffit for achieving higher fire resistance.

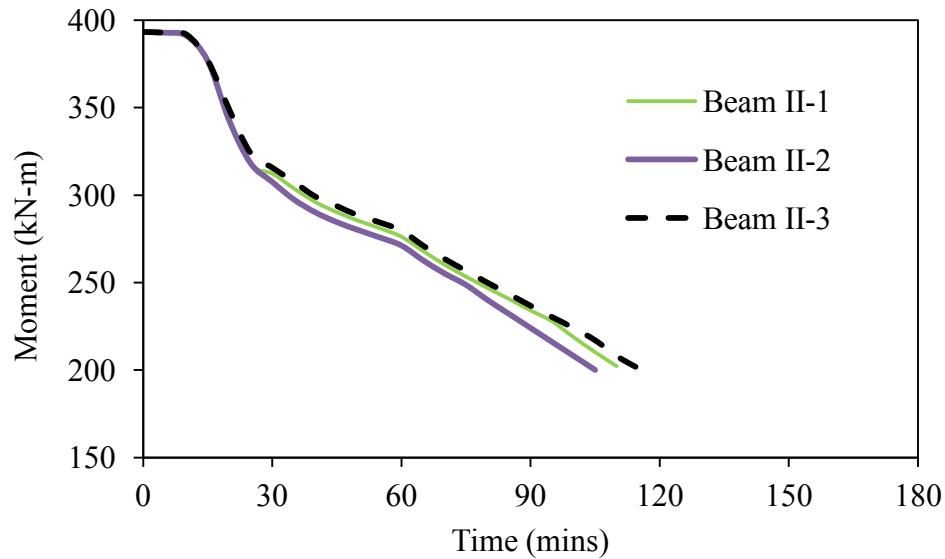


Figure 6.9 Effect of FRP location on the variation of moment capacity of NSM strengthened RC beams

6.4.3 Effect of reinforcement ratio of FRP and steel rebar

The flexural strength in NSM FRP strengthened RC beams is provided by steel rebars and NSM FRP reinforcement. Under fire conditions, since NSM FRP reinforcement experiences much faster degradation in strength and stiffness properties, steel rebars contribute to higher percentage of flexural capacity at higher temperatures.

Thus, reinforcement ratio of steel rebar to that of NSM FRP influences the fire resistance of NSM FRP strengthened RC beams.

Since tensile strength and elastic modulus of NSM FRP and steel rebar are quite different, a comparison of product of rebar area and tensile strength of NSM FRP and steel rebar is a better measure to reflect the relative contribution of these two reinforcement to flexural capacity. Thus, this parameter (product of rebar area and tensile strength) was varied over a practical range in the analysis to study the influence of reinforcement ratio of NSM FRP and steel rebar. As shown in Table 6.4, Beam III-1 is reinforced with four ϕ 19 mm steel rebars and four 16×4 mm FRP strips, and tensile strength of steel and FRP is 576 and 2068 MPa respectively. Beam III-2 has the same size of steel rebars and FRP strips, but tensile strength of steel rebar is 461 MPa (80% of 576 MPa), so the reinforcement ratio of steel rebar is actually lower than Beam III-1. Beam III-3 has the same tensile strength of steel and FRP, but the area of FRP strips is 16×2 mm, and thus the reinforcement ratio of steel rebar is higher than that of Beam III-1. The characteristics of NSM FRP and steel rebar, and moment contribution of steel and FRP for these three beams are compared in Table 6.4.

Table 6.4 Configuration and moment capacity contribution of NSM FRP
and steel rebar in Beams III 1-3

Beams	Ratio of product of area and strength			Room temperature			Fire resistance (mins)
	NSM FRP	Steel rebar	FRP /steel	Moment capacity (kN-m)	FRP contribution	rebar contribution	
Beam III-1	4 16×4mm strips $f_f=2068\text{MPa}$	4 ϕ 19 mm rebars $f_y=576\text{MPa}$	0.81	393	38.7%	61.3%	110
Beam III-2	4 16×4mm strips $f_f=2068\text{MPa}$	4 ϕ 19 mm rebars $f_y=461\text{MPa}$	1.01	325	41.5%	58.5%	80
Beam III-3	4 16×2mm strips $f_f=2068\text{MPa}$	4 ϕ 19 mm rebars $f_y=576\text{MPa}$	0.41	333	29.1%	70.9%	135

The above three beams were analyzed under the same level of applied loading, (50% of room temperature moment capacity) to demonstrate the influence of FRP to steel reinforcement ratio on fire resistance of NSM FRP strengthened RC beams. The thermal response of three beams is identical, due to the same locations of steel rebar and FRP strips. The structural response of these beams is compared in Figure 6.10, by plotting the variation of moment capacity with fire exposure time. It can be seen in Figure 6.10 that Beams III-2 and III-3 possess similar flexural moment capacity at ambient conditions. However, under fire conditions, the moment capacity of Beam III-2 decreases much faster than that of Beam III-3. This is mainly attributed to the fact that NSM FRP strips in Beam III-2 lose strength rapidly. Therefore, the contribution of FRP strips to moment capacity (41.5%) decreases significantly. While in Beam III-3, NSM FRP strips contribute less percentage to moment capacity (29.1%). Even though FRP strips lose most of strength, the major part of moment capacity, which is provided by steel rebars,

decreases at a very low pace. Analysis results indicate that Beam III-2 fails at 80 minutes (fire resistance), while Beam III-3 fails at 135 minutes. Beam III-1 has a moderate reinforcement ratio of FRP (38.7%), and thus fails at 110 minutes. Based on comparative response of three beams, higher reinforcement ratio of NSM FRP leads to lower fire resistance in NSM FRP strengthened RC beams.

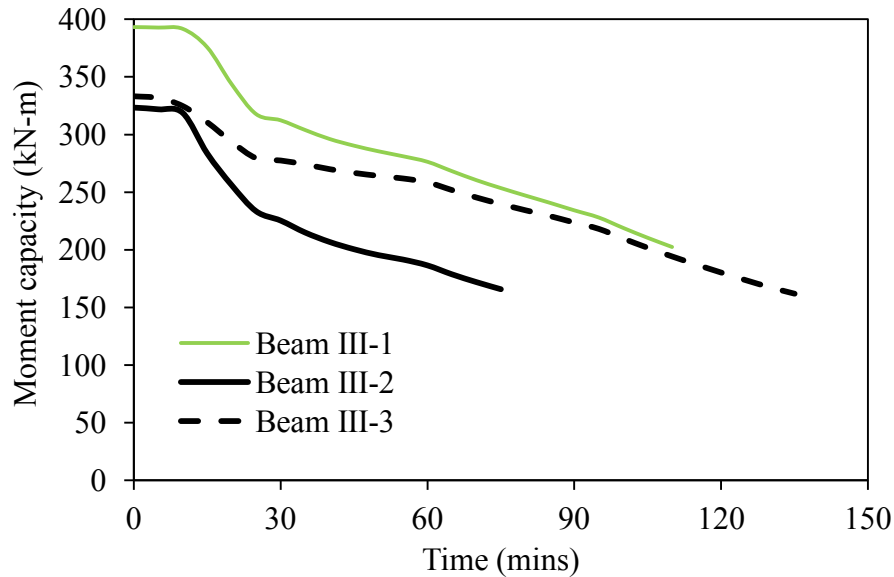


Figure 6.10 Effect of reinforcement ratio of FRP and steel rebar on the variation of moment capacity of NSM strengthened RC beams

6.4.4 Effect of concrete compressive strength

The effect of concrete compressive strength on fire response of NSM FRP strengthened RC beam is evaluated through fire resistance analysis on beams with concrete strength ranging from 35-60 MPa. In the analysis, the temperature dependent thermal and mechanical properties of concrete are assumed to follow those of normal strength concrete as recommended in Eurocode 2 (2004). This study is limited to the

scope of normal strength concrete, so any fire induced spalling is not included in the analysis. Therefore, thermal response of these beams is the same throughout fire exposure.

Figure 6.11 shows the effect of compressive strength of concrete on fire response of NSM FRP strengthened RC beams, where variation of moment capacity is plotted against fire exposure time. It can be seen that the room temperature capacity of strengthened beams increases with the increase in concrete strength. This is due to the fact that NSM FRP strengthened beams usually fail in compression zone, and higher concrete strength helps in utilizing more strength of NSM FRP strips. However, at high temperatures, the effect of concrete strength becomes minor, since high temperature moment capacity of strengthened beams mainly depends on degradation of strength in steel rebar and FRP strip. As mentioned earlier, thermal response of the beams is same. Therefore, steel rebar and FRP strips retain the same strength and stiffness, the moment capacity of these beams also decreases in a similarly pattern. Overall, the results from this analysis indicate that compressive strength of concrete is not a critical factor influencing fire response of NSM FRP strengthened RC beams.

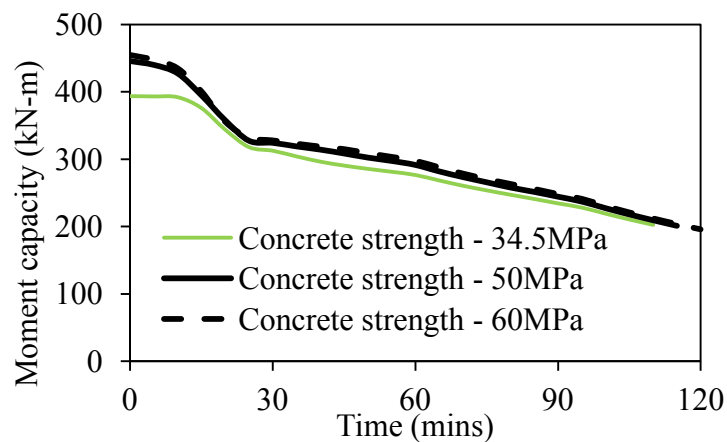


Figure 6.11 Effect of concrete compressive strength on the variation of moment capacity of NSM strengthened RC beams

6.4.5 Effect of load level

The level of loading on RC beam can significantly influence the structural behavior under fire conditions. Higher load level leads to quick degradation in flexural capacity and stiffness of beam, and introduces larger deflection in beam. Also RC beams can fail before strength limit state is reached, due to exceeding deflection limits. In this section, numerical analysis was carried out on an NSM FRP strengthened RC beam under 40, 50, 60 and 70% load ratios, to study the influence of load level. The load ratio is calculated as the ratio of bending moment due to applied load under fire conditions to room temperature nominal capacity of the beam. The effect of load ratio on the fire response of NSM FRP strengthened RC beam is illustrated in Figures 6.12.

Since material properties and heat transfer within beam cross-section are not affected by load level, the thermal response of these beams remains identical. However, load ratio significantly influences fire resistance of NSM FRP strengthened RC beam based on strength capacity and deflection criteria. Overall the fire resistance decreases with increasing load ratio. This can be attributed to the fact that beams at higher load levels experience higher internal stresses, and this in turn leads to significant degradation in strength and stiffness properties of constitutive materials. Therefore, the beams with higher load ratios (more than 50%) experience much larger deflections, as shown in Figure 6.12. Finally the beams fail under fire conditions due to reaching the moment capacity limit.

Also, relatively higher load level produces large curvatures in the beam, and thus NSM FRP reinforcement fails earlier than the beams under smaller loading. This also accelerates the failure of NSM FRP strengthened beams under fire conditions. Thus, in

structural fire design of NSM FRP strengthened beams, the influence of loading level needs to be accounted for, and load ratio under fire conditions should be limited to 50%.

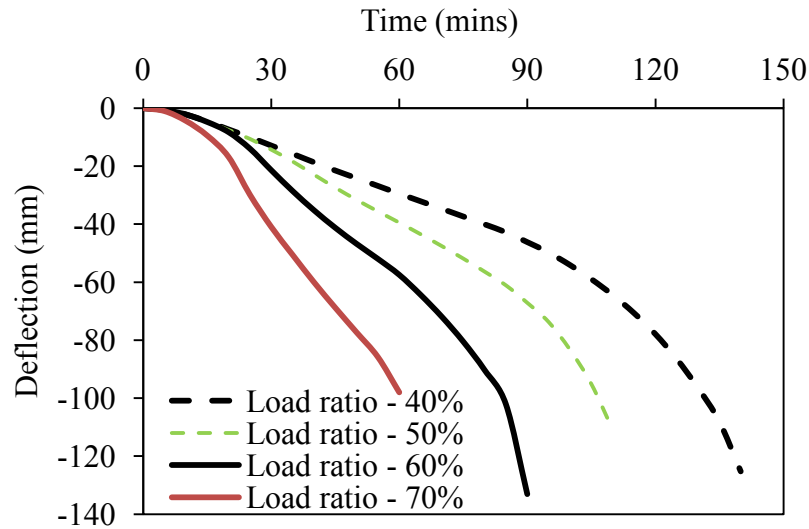


Figure 6.12 Effect of load level on the variation of mid-span deflections of NSM strengthened RC beams

6.4.6 Effect of axial restraint

Reinforced concrete beams in buildings are generally subjected to some level of axial restraint from adjoining frame members. The restraint can have significant influence on fire response of RC beams (Dwaikat and Kodur 2008). Therefore, it is important to account for the influence of axial restraint in evaluating fire resistance of NSM FRP strengthened RC beam. To study this influence, a typical RC beam strengthened with NSM FRP strips (Beam VI-1) is analyzed with different levels of axial restraint at the beam ends. The axial restraint stiffness is varied over a range that covers typical boundary conditions encountered in buildings. For instance, a stiffness of 5 kN/mm represents the restraint provided by a typical beam column connection, while a stiffness

of 200 kN/mm represents the restraint provided by a shear wall (Dwaikat and Kodur 2008). In the analysis, the beam is exposed to ASTM E119 standard fire and subjected to a uniformly distributed load, which corresponds to 50% of room temperature capacity.

Fire response of NSM FRP strengthened RC beams with axial restraint is illustrated in Figure 6.13, by plotting mid-span deflection in beam as a function of fire exposure time. It can be seen that the beam with larger axial restraint stiffness undergoes smaller deflection under fire exposure, and also yields higher fire resistance. This high fire resistance can be attributed to fire-induced axial forces that develop at the beam ends. When exposed to fire, a concrete beam undergoes thermal expansion along the beam length, and then an axial restraint forces (compression) get developed at the ends of the beam. Since this fire induced axial force acts at the mid-depth of cross-section, which is at a lower level than the location of neutral axis of beam, axial force will produce a counter acting moment and the total moment applied on the beam is reduced, as illustrated in Figure 6.14. Correspondingly, the deflection of beam also decreases, due to fire induced axial force. Larger restraint stiffness produces larger axial force and larger counter acting moment, and thus the deflection is even smaller and the fire resistance is further enhanced.

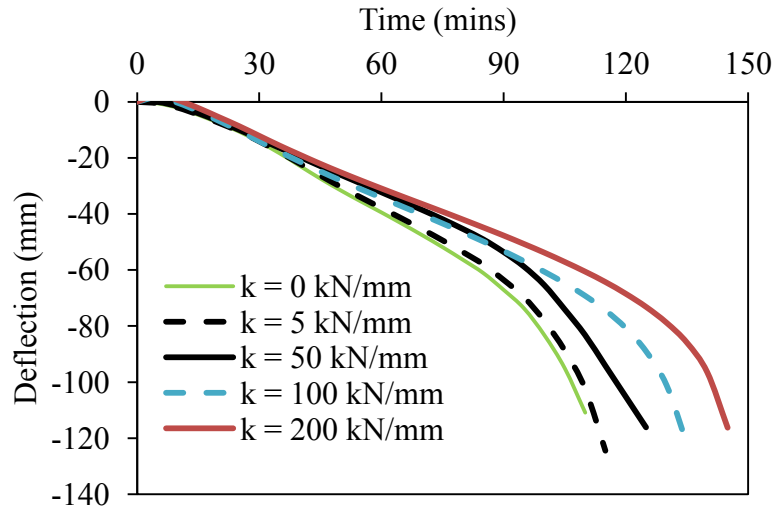


Figure 6.13 Effect of axial restraint on the variation of mid-span deflections of NSM FRP strengthened RC beams

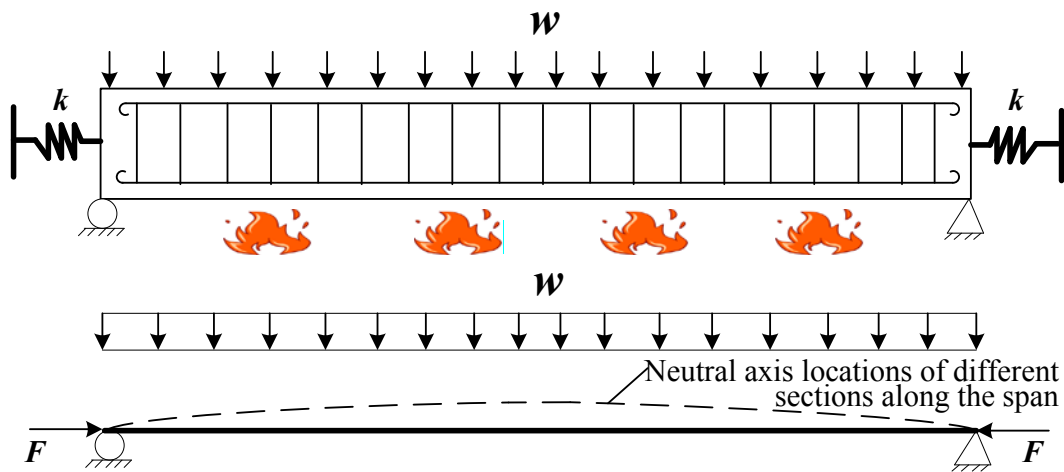


Figure 6.14 Illustration of axial restraint force under fire conditions

As shown in Figure 6.13, the mid-span deflections in different cases are quite close at the initial stages of fire exposure, this is due to similar strength degradation in FRP strips and steel rebars in each beam. After 45 minutes into fire exposure, the simply supported beam ($k = 0$ kN/mm) experiences larger deflection, while the beams with axial restraint undergo smaller deflections due to the effect of counter-moment. After 90

minutes, all beams start to experience accelerating deflections, due to significant strength and stiffness loss in steel rebar. Analysis results indicate that the simply supported beam achieves a fire resistance of 110 minutes, while the beams with axial restraint achieve higher fire resistance by about 5-30 minutes, as shown in Figure 6.13.

The development of axial restraint force in beams with different levels of restraint stiffness is plotted in Figure 6.15, as a function of fire exposure time. As expected, higher axial restraint stiffness produces higher axial force throughout fire exposure duration. It can be seen that at the initial stage of fire exposure, the axial forces increase at a faster rate than that at later stages. This can be attributed to rapid thermal expansion along the beam in the early stage. However, after 60 minutes into fire exposure, there is no major increase in axial forces. This is mainly due to temperature induced degradation in strength and stiffness of constituent materials (concrete, steel rebars and FRP strips), and this in turn leads to decrease of beam stiffness. With the increase of mid-span deflections (critical section) under fire conditions, the neutral axis of beam moves down and the counter-moment generated from axial restraint also reduces. Therefore, the axial restraint forces have marginal benefit at the later stages of fire exposure.

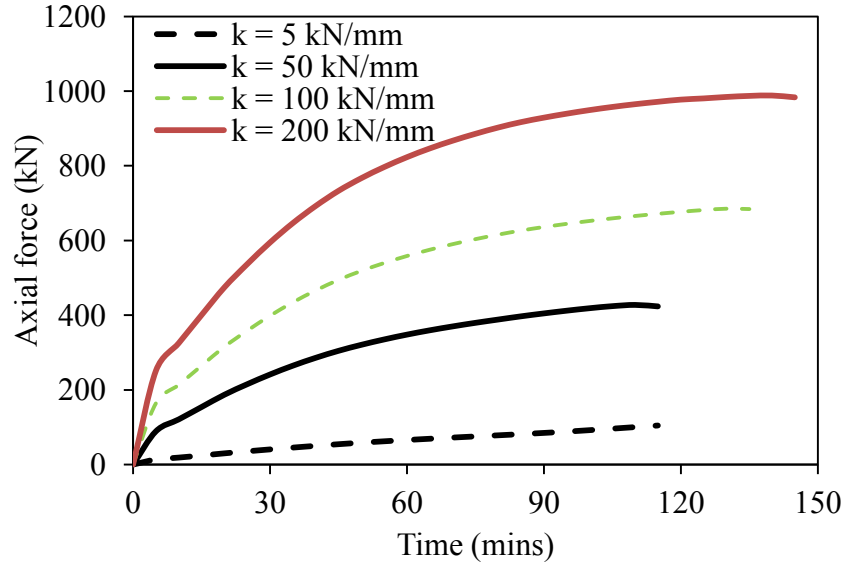


Figure 6.15 Variation of axial force in NSM FRP strengthened RC beams as a function of fire exposure time

ACI 216.1 specifications (2007) also indicates that restrained concrete beams can achieve higher fire resistance than that of unrestrained beams, yet the extent of influence of restraint conditions is not fully quantified. The numerical studies herein quantify the increase in fire resistance for strengthened RC beams under various levels of axial restraint.

6.4.7 Effect of fire scenario

Previous fire resistance tests on RC beams strengthened with NSM FRP were mainly evaluated under standard fire exposure, which may not represent the true response of these beams under realistic fire conditions. Thus, an NSM strengthened RC beam was analyzed under various fire exposure conditions to evaluate the effect of fire scenarios on fire resistance. Figure 6.16 shows time-temperature curves for different fire scenarios used in the analysis, which are ASTM E119 standard fire, ASTM hydrocarbon fire, and

two design (realistic) fires. The time-temperature relations of two design fires are generated based on Eurocode 1 provisions (Eurocode 1 2002), and they represent a wide range of compartment characteristics including fuel load and ventilation. Design fire I represents a severe fire in a library or a storage room with sufficient ventilation and a large amount of combustible material. The peak temperature is assumed to be 1250°C, and then the decay phase lasts for 200 minutes. Design fire II represents a typical fire in a residential compartment. The NSM FRP strengthened beam in the analysis is unprotected (no fire insulation), and it is subjected to a uniformly distributed load along the whole span.

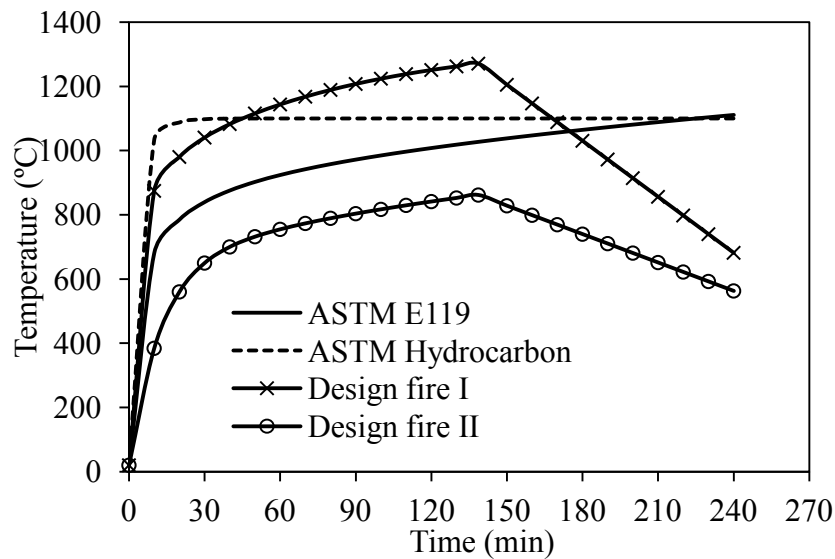


Figure 6.16 Standard and design fire temperature curves used in parametric study

The comparative thermal response of RC beam under different fire scenarios is shown in Figure 6.17, by plotting the temperature in corner FRP strips as a function of fire exposure time. It can be seen that the temperature rise in FRP strips highly depends on the type of fire exposure. After 60 minutes into fire, FRP strips under exposure to hydrocarbon fire and Design fires I attain much higher temperature than that under

ASTM E119 fire and Design fire II, due to much higher severity of these two fires. FRP temperature under ASTM Hydrocarbon fire keeps increasing during the entire of fire exposure, while the one under Design fire I starts to decrease in the later stage of fire. For Design fire I, once the decay phase starts, the heat propagation within beam ceases, and thus the rise in rebar temperatures gradually stops and starts to decrease. However, under Design fire I, FRP strips already went beyond 800°C prior to the decay phase, which infers that FRP strips lost most of its strength and stiffness before the start of the decay phase.

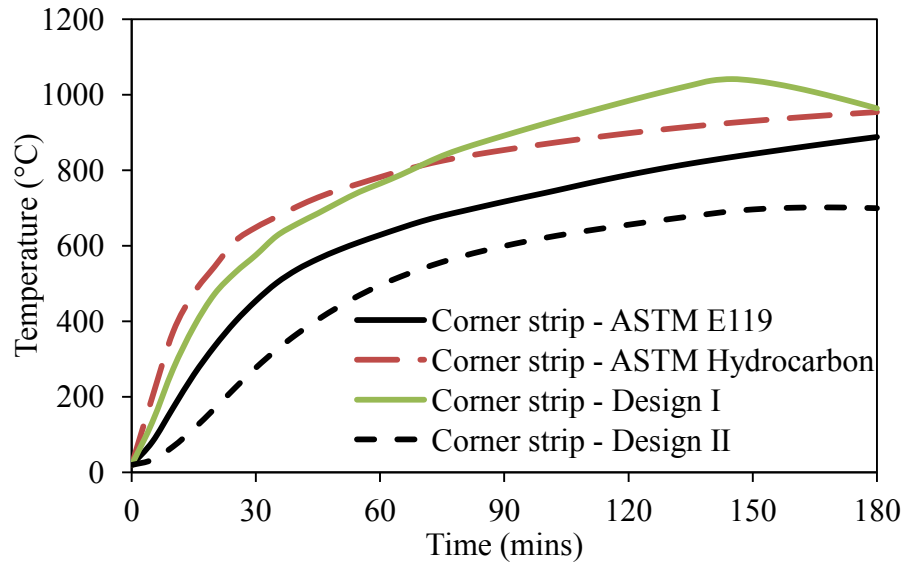


Figure 6.17 Effect of fire exposure on temperature rise in corner FRP strip

To compare structural response of the above beams under different fire scenarios, the variation of mid-span deflection with fire exposure time is plotted in Figure 6.18. The mid-span deflections in hydrocarbon fire and Design fires I (severe fires) are similar, and they experience much faster rise than the other two cases. This is mainly due to faster degradation of strength and stiffness of steel rebar and FRP strips in these severe fires. It can be seen that the strengthened beam achieves a fire resistance of about 75 minutes

under severe fire, and the failure is governed by strength limit criteria. However, under moderate fire (Design fire II), the deflection rate of the beam is relatively small, and the beam has in a low deflection for 120 minutes. This is mainly due to high retention of beam stiffness resulting from relatively slower heat propagation within beam cross-section. These results indicate that NSM strengthened RC beams can achieve high fire resistance of 150 minutes under moderate fire exposure conditions, even without any fire insulation.

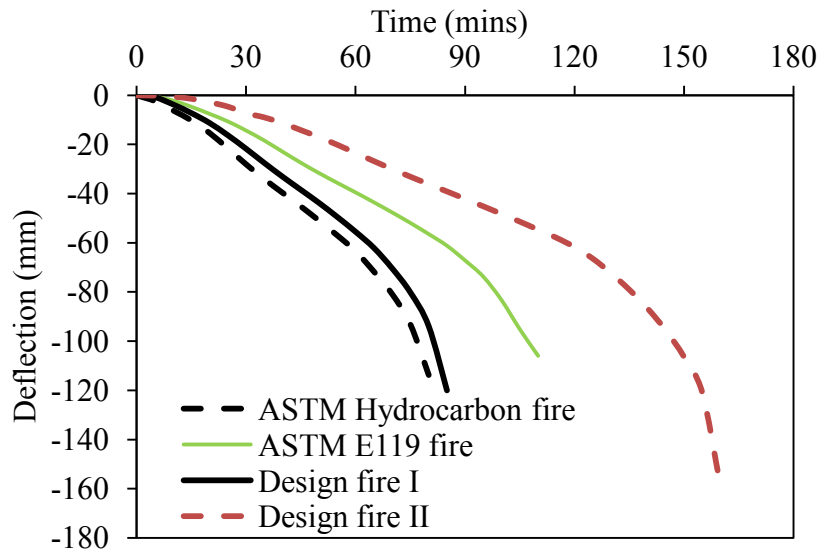


Figure 6.18 Effect of fire exposure on the variation of mid-span deflections in NSM FRP strengthened RC beams

6.4.8 Effect of insulation layout

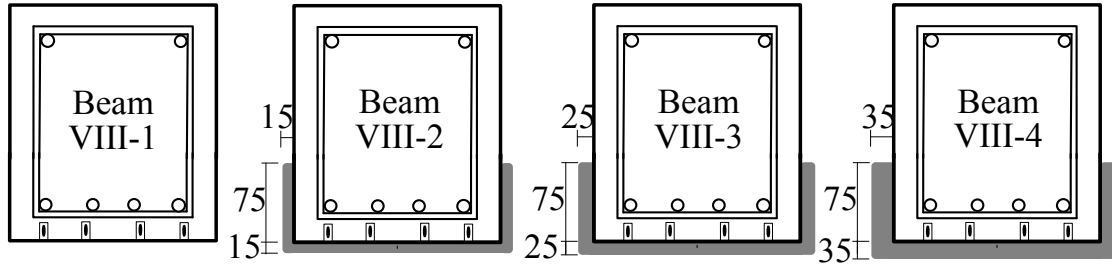
Results from the above analysis clearly indicate that NSM FRP strengthened RC beam possess a relatively higher fire resistance than that of externally bonded FRP beams. However, structural members in building might need to provide up to four hours of fire resistance, depending on occupancy and importance of the building. Thus fire insulation

has to be applied on NSM FRP strengthened RC beam. To develop optimum fire insulation schemes, a parametric study was carried out on a typical NSM FRP strengthened RC beam (Beam VIII-1) with various insulation schemes. The thermal properties of fire insulation are assumed to follow those of VG insulation reported by Bisby (2003) and included in the Appendix A.

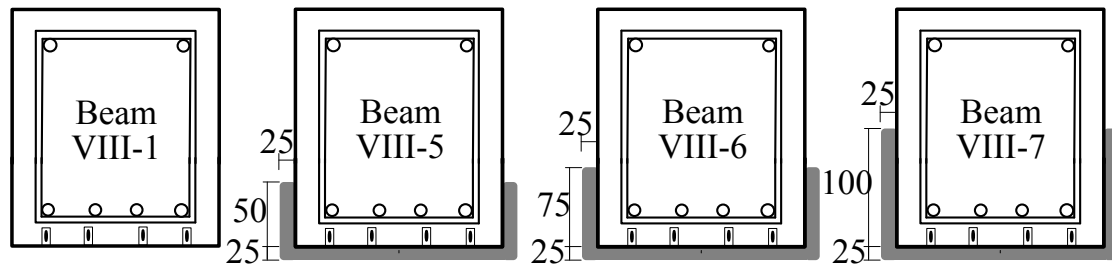
When an RC beam is exposed to fire, temperature-induced strength and stiffness degradation in FRP and steel reinforcement (tension) dominates the response of strengthened beam. Thus, the best way to enhance fire performance is to provide optimum fire protection on the bottom and side surfaces of the beam. Thus insulation thickness and depth of insulation on side surfaces are the two most important parameters. Two sets of analysis are carried out on Beam VIII-1 to study the effect of insulation (See Figure 6.19). In the first set, the insulation thickness at the beam soffit and two sides (Beam VIII-1) were varied from 15 to 35 mm, while the depth of insulation on two sides of beam were kept constant at 75 mm (three times of concrete cover thickness). In the second set, the depth of insulation on side surfaces was varied from 50 to 100 mm, and the thickness of insulation on side and bottom surfaces of beam was kept constant at 25 mm. The detailed information on insulation layout is shown in Figure 6.19 and Table 6.5.

Table 6.5 Effect of insulation layout on fire response of NSM FRP strengthened beams

Factors in study	Beams	Insulation thickness (mm)	Insulation depth (mm)	Fire resistance (mins)
Uninsulated beam	Beam VIII-1	--	--	110
Effect of insulation thickness	Beam VIII-2	15	75	190
	Beam VIII-3	25	75	235
	Beam VIII-4	35	75	255
Effect of insulation depth	Beam VIII-5	25	50	200
	Beam VIII-6	25	75	235
	Beam VIII-7	25	100	250



(a) Beams analyzed for evaluating the effect of insulation thickness



(b) Beams analyzed for evaluating the effect of insulation depth

Figure 6.19 RC beams analyzed for studying the effect of fire insulation scheme

The analysis indicates that steel rebars in the insulated beam remain lower than 400°C and do not lose strength in most fire duration. Thus the temperature in NSM FRP has dominant influence on the fire response of the strengthened beam. Figure 6.20 illustrates the effect of insulation thickness on the temperature in corner NSM FRP strips. The temperature rise is plotted against fire exposure time for four cases, one without any insulation and the other three with varying insulation thickness at beam soffit. It can be seen that the application of insulation significantly slows down the temperature rise in NSM FRP strip, and this in turn leads to slower degradation of strength and stiffness in NSM FRP strip. When the thickness of insulation increases from 15 to 25 mm, the temperature in NSM FRP decreases significantly at any given fire exposure time. Even after 4 hours of fire exposure, the temperature in NSM FRP remains below 600°C . This indicates for the most fire durations, NSM FRP possesses some level of strength and

contributes to moment capacity of beam. However, when the thickness of insulation increases from 25 to 35 mm, there is no major advantage (rebar temperature or fire resistance) from added insulation, since fire resistance of the insulated beam is almost four hours of fire exposure and this is sufficient for meeting needed fire resistance in buildings. This analysis infers that beyond an optimum insulation thickness, increasing insulation thickness does not help in achieving any significant increase in fire resistance, as indicated in Table 6.5. Therefore, for this type of fire insulation and beam configuration, an optimum thickness of fire insulation is 25 mm.

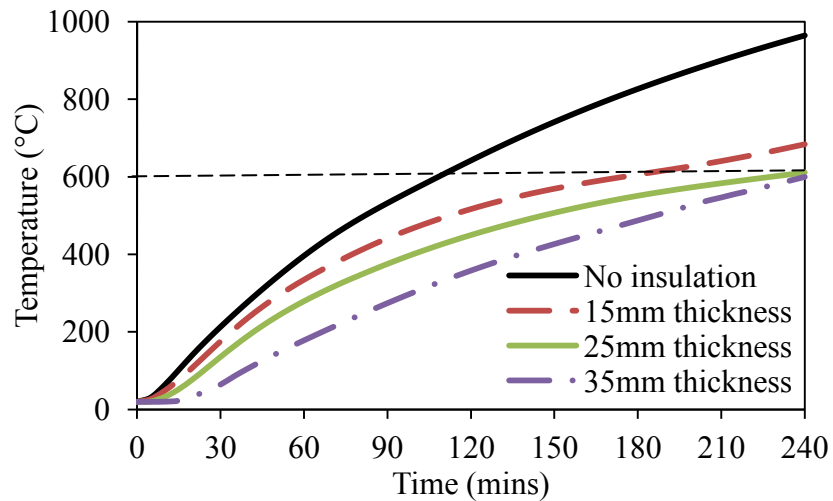


Figure 6.20 Effect of insulation thickness on temperature rise in NSM FRP strips

The above analysis was also applied to Beam VIII-1 by varying depth of insulation on side surface (c) of the beam from 50 mm to 100 mm, and keeping the insulation thickness constant at 25 mm. The variation of predicted temperatures in corner NSM FRP strips are plotted in Figure 6.21. It can be seen that increasing depth of insulation from 50 mm to 75 mm lowers corner FRP temperatures by about 80°C (at 4 hours fire exposure), while further increasing in the depth of insulation to 100 mm does not lower NSM FRP temperature significantly. This is because the 75 mm depth of

insulation is three times of concrete cover and it is high enough to decrease the heat transfer from two sides of beam. Higher depth of insulation does not produce more protective effect. From the point of view of structural response (see Table 6.5), the beam with 75 mm depth of insulation on side surfaces achieved a fire resistance higher than the case of 50 mm depth of insulation. While beyond 150 mm, the increase in fire resistance is marginal (only 15 minutes). Therefore, an optimum insulation scheme is a U-shaped insulation which comprises of 25 mm insulation in thickness and 75 mm in depth on two sides of beam. This analysis clearly illustrates usefulness of the numerical model in developing an optimum insulation scheme for NSM FRP strengthened RC beams.

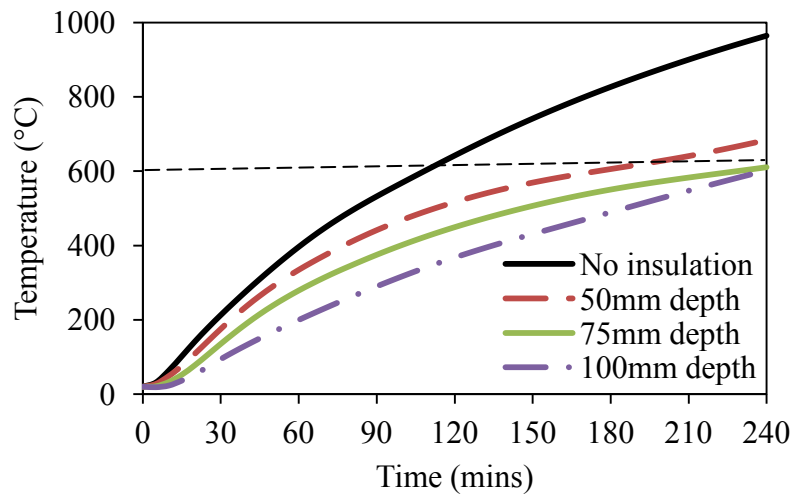


Figure 6.21 Effect of insulation depth on temperature rise in NSM FRP strips

6.5 Summary

This chapter presents results of parametric studies on critical factors influencing fire response of NSM FRP strengthened RC beams. Based on the analysis results, FRP strengthening type, reinforcement ratio of steel and FRP, load level, axial restraint, fire

scenario, and insulation scheme have significant influence on the fire resistance of NSM FRP strengthened RC beam. While NSM FRP location and concrete strength have moderate influence on fire resistance. The results obtained in parametric studies helps to develop design guidelines for fire resistance of NSM FRP strengthened RC beams. However, since geometric and material properties of FRP strengthened RC beams vary in a wide range, a simple and reliable design approach is needed to access fire resistance of various NSM FRP strengthened RC beams. Development and verification of such design methodology are presented in Chapter 7.

CHAPTER 7

RATIONAL DESIGN METHODOLOGY

7.1 General

As discussed in Chapter 2, there are only limited design guidelines on fire design of FRP strengthened RC members. These design guidelines recommend neglecting the contribution of FRP to capacity under fire conditions (ACI 440.2 2008, FIB Bulletin 14 2007). However, results from previous studies clearly indicate this assumption to be over conservative, especially for NSM FRP strengthened RC beams and beams protected with fire insulation. Thus, rational design methodology is needed for evaluating fire response of FRP strengthened RC beams.

To overcome this drawback, a simplified approach is developed for predicting the fire response of FRP strengthened RC beams. This approach is derived by applying current fire design approach for RC beams but incorporating the effect of FRP and fire insulation in fire resistance calculations. The proposed approach comprises of two main steps, namely; evaluating temperatures in concrete, FRP, and steel rebar of strengthened RC beams, and calculating moment capacity of FRP strengthened RC beam at a given fire exposure time. Simplified equations are developed for evaluating temperatures in concrete, steel rebar, and FRP reinforcement (NSM or EBR) and then procedure for calculating moment capacity of FRP strengthened RC beams is outlined.

The validity of the proposed simplified approach is established by comparing predicted fire response parameters with those obtained from fire resistance tests and

detailed numerical studies. The applicability of this approach is also illustrated through practical design examples.

7.2 Simplified Approach for Predicting Temperatures in RC Members

Fire resistance of FRP strengthened RC beams is mainly influenced by temperature-induced strength degradation in FRP, steel rebar, and concrete. Thus, accurate estimation on cross-sectional temperature is a key step in evaluating fire response of FRP strengthened RC beams.

For an FRP strengthened RC beam without any external fire insulation, temperature progression within the beam cross-section can be taken to be very much similar to that of original RC beam, since the amount of FRP (either NSM FRP or external FRP) is too small to influence heat transfer in an RC member. However, in an FRP strengthened RC beam with fire insulation, temperature progression in concrete, steel and FRP reinforcement is significantly reduced by insulation layer, and thus cross-sectional temperatures within concrete beam are much lower than those of an uninsulated beam. Therefore, separate equations are needed for evaluating temperatures in uninsulated and insulated beam respectively. Detailed procedure for developing these simplified equations is presented in this section.

7.2.1 An approach for predicting temperature in an uninsulated RC member

7.2.1.1 General

For predicting temperature in a fire exposed RC member, a number of design graphs and charts are available in codes and standards (ACI 216 2007, Eurocode 2 2004).

However, these graphs usually provide very conservative temperature predications, as these are developed based on standard fire test data or results of analysis on specific types of concrete (Kodur et al. 2013). Also, in both ACI 216.1 and Eurocode 2, the temperature profiles are provided in time intervals of 0.5 hour or 1 hour, so an interpolation is required for evaluating temperature at other time intervals, which might increase inaccuracy of temperature predictions.

A review of literature indicates that there are few simplified expressions for evaluating the cross-sectional temperature in an RC member as a function of depth and fire exposure time (Hertz, K. 1981, Wickstrom 1986). The most notable of these simplified temperature expressions is the empirical relation proposed by Wickstrom, for normal-weight aggregate concretes (Wickstrom 1986). According to Wickstrom's method, the temperature (T_c) in an RC member (slab) exposed to fire from one side at a given depth x (in meters) and time t_h (in hours) can be calculated as:

$$T_c = \eta_x \cdot \eta_w \cdot T_f \quad (7.1)$$

$$\eta_x = 0.18 \ln(t_h / x^2) - 0.81 \quad (7.2)$$

$$\eta_w = 1 - 0.0616 t_h^{-0.88} \quad (7.3)$$

In Eqns. 7.1-7.3, η_w is the ratio between concrete surface temperature and the fire temperature, η_x is the heat transfer factor induced through one fire-exposed surface, and T_f is the fire temperature. For obtaining temperatures at corner locations of a structural member, the heat conduction occurring in two directions (x and y) is to be accounted for

through η_x and η_y , in which η_y is calculated similar to η_x in Eq. 7.2. The temperature T_c resulting from fire exposure from both x and y directions is

$$T_c = [\eta_w(\eta_x + \eta_y - 2\eta_x\eta_y) + \eta_x\eta_y]T_f \quad (7.4)$$

where, η_x and η_y are the heat transfer factors induced by each side of fire exposure.

Wickstrom's empirical equation provides only a rough estimation of cross-sectional temperatures, since it does not account for different growth rates of fire temperatures (Buchanan 2002). Also, this equation does not include the influence of aggregate type (siliceous or carbonate), newer concrete types (high strength concrete) and variation of thermal properties with temperature, and hence may not be applicable for different concrete types (Kodur et al. 2013). Therefore, there is a need for simple and reliable approach for evaluating cross-sectional temperatures in a fire-exposed RC member.

Unlike the case of steel structural members, the temperature profiles in RC members have a large spatial variation within the cross section. This makes it quite complex to derive a simplified expression for temperatures formula based on heat transfer principles. Developing an expression based on nonlinear regression analysis on a database of cross-sectional temperatures is a more feasible approach, and the procedure of developing such expression is outlined below.

7.2.1.2 Generation of temperature data for regression analysis

The simplified equations for predicting temperatures in an RC member are developed through a nonlinear regression analysis on a large database generated using finite element analysis (FEA). To generate temperature data for regression analysis,

twenty representative RC beams are analyzed using the finite element program described in Chapter 5. Since temperature rise in fire-exposed RC members is mainly influenced by cross-sectional geometry, concrete strength, aggregate type and fire exposure conditions (Thomas and Webster 1953, Ali et al. 1996, Kodur et al. 2004), selected RC beams are varied over a wide range and the ranges of these parameters are tabulated in Table 7.1.

As shown in Table 7.1, the width of beam section was varied from 200 to 500mm, while the depth was varied from 400 to 700mm, thus giving a width to depth ratio ranging from 0.4 to 0.8. The twenty beams were classified into four groups, Group I to Group IV, to account for the effect of different concrete types. Group I and Group III were assumed to be made of normal strength concrete (NSC), while Group II and IV were assumed to be made of high strength concrete (HSC). Further, Group I and II were of carbonate aggregate (CA) concrete, while Group III and IV were of siliceous aggregate concrete (SA). Temperature dependant thermal conductivity and thermal capacity of these concretes vary according to the relations given in ASCE manual (Lie 1992) or by Kodur et al. (2008). In the FEA, each beam was exposed to ASTM E119 fire exposure from three sides for 4 hours, and the cross-sectional temperatures at 30 locations were recorded at various time intervals. In total, 1200 temperature data points with corresponding time and location information ($5 \times 30 \times 8$) were generated for each group of beams.

Table 7.1 Characteristics of RC members for regression analysis

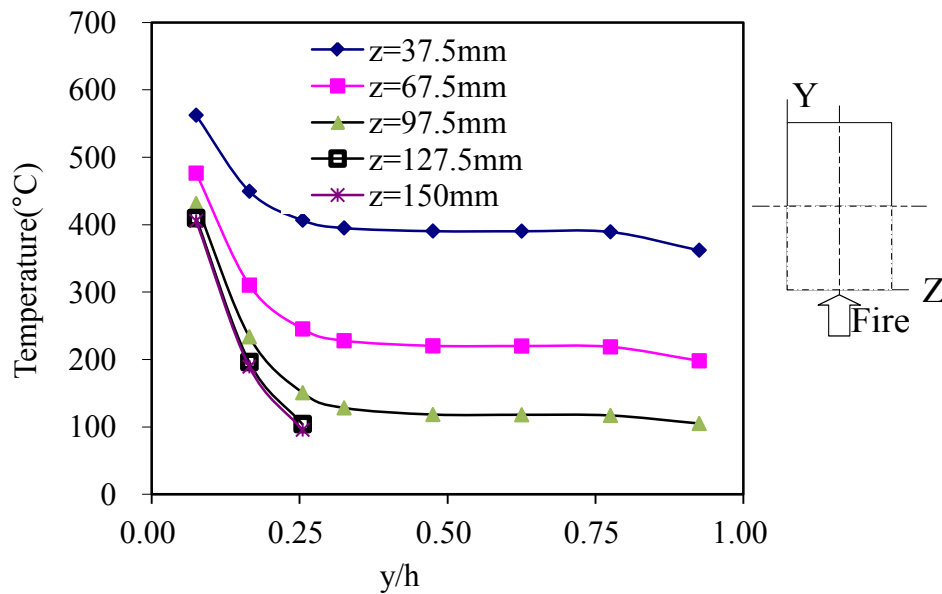
Group	Concrete Strength	Aggregate	Cross-sectional dimensions	
			Dimension of beam(mm)	b/h
I	NSC	Carbonate	200×500	0.4
			300×400	0.75
			300×500	0.6
			300×700	0.43
			400×500	0.8
II	HSC	Carbonate	200×500	0.4
			300×400	0.75
			300×500	0.6
			300×700	0.43
			400×500	0.8
III	NSC	Siliceous	200×500	0.4
			300×400	0.75
			300×500	0.6
			300×700	0.43
			400×500	0.8
IV	HSC	Siliceous	200×500	0.4
			300×400	0.75
			300×500	0.6
			300×700	0.43
			400×500	0.8

7.2.1.3 Cross section division for 1-D and 2-D heat transfer area

Due to the complexity associated with 3D heat transfer analysis, and variation of thermal properties of concrete with temperature, the cross-sectional temperatures are often evaluated based on 1-dimensional (slabs) or 2-dimensional (beams or columns) heat transfer analysis. However, there is no specific criteria for categorizing 1-D or 2-D heat transfer area division within a concrete member.

A parametric study was conducted to investigate the relationship between temperatures and distance from each fire-exposed beam surface individually. Figure 7.1 illustrates the variation of cross-sectional temperatures with depth to the bottom (y) at

different fire exposure times. It can be seen that at a given width (z), the cross-sectional temperature decreases dramatically with increasing depth (y) up to the mid-depth of the section ($h/2$). Beyond half of beam depth ($y/h > 0.5$), the temperatures barely increase with y and almost remain constant. This means the heat transfer from the bottom mainly influences the temperatures in the lower half of the beam, but has little effect on the temperatures in the upper half of the beam. In the horizontal direction (z), the temperature decreases with the distance from the surface (z) significantly until z/b approaches to 0.5, as shown in Figure 7.2. This indicates the heat transfer from the surface of fire exposure mostly influences temperature in half of depth or width of the beam till three or four hours of fire exposure.



(a) 1.5 hours

Figure 7.1 Variation of temperature with depth from the bottom of an RC beam at various times (section 300×500mm)

Figure 7.1 (cont'd)

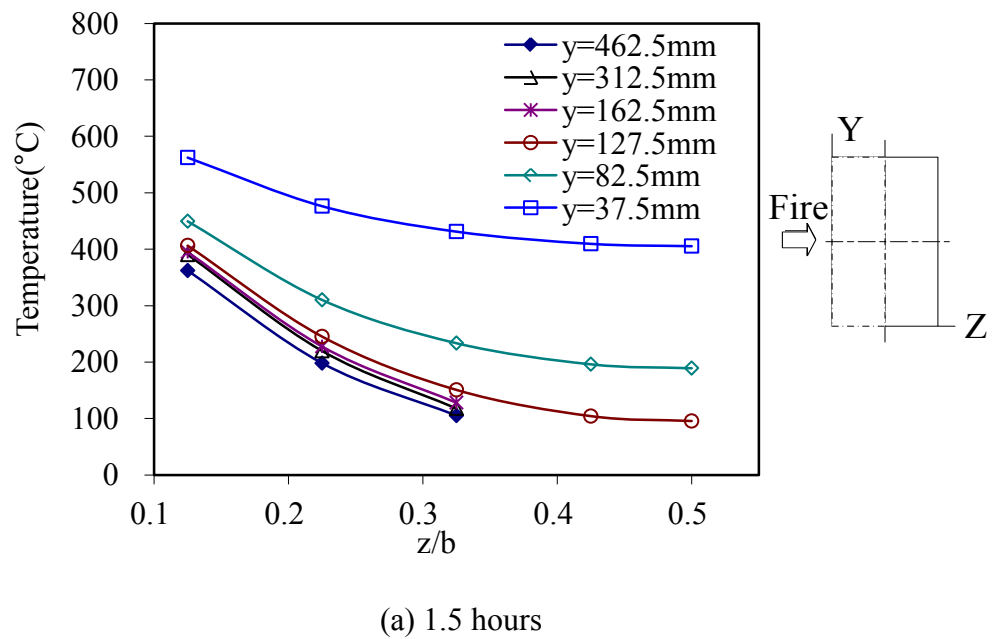
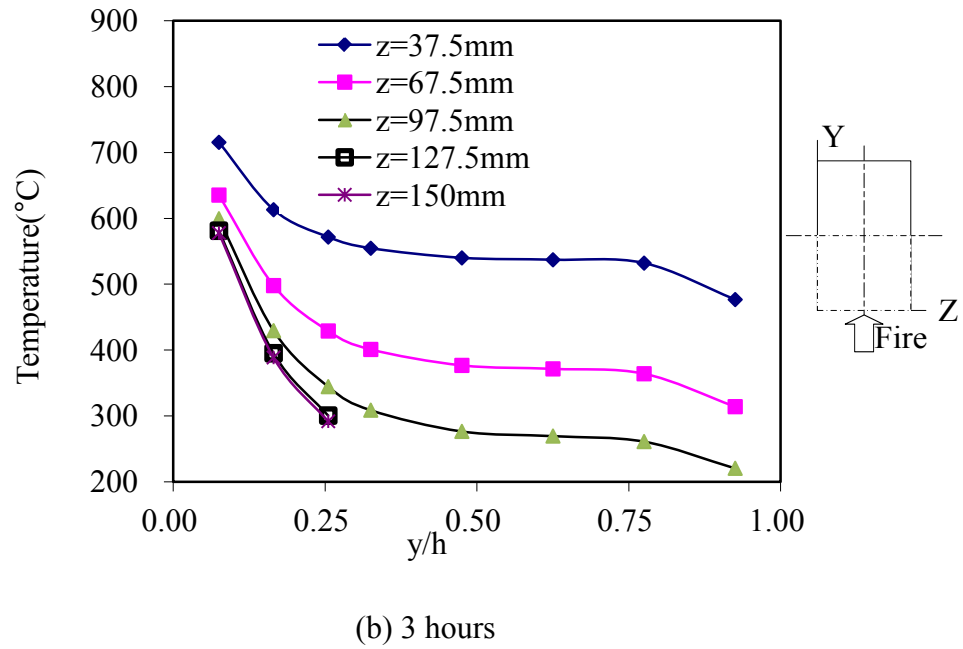
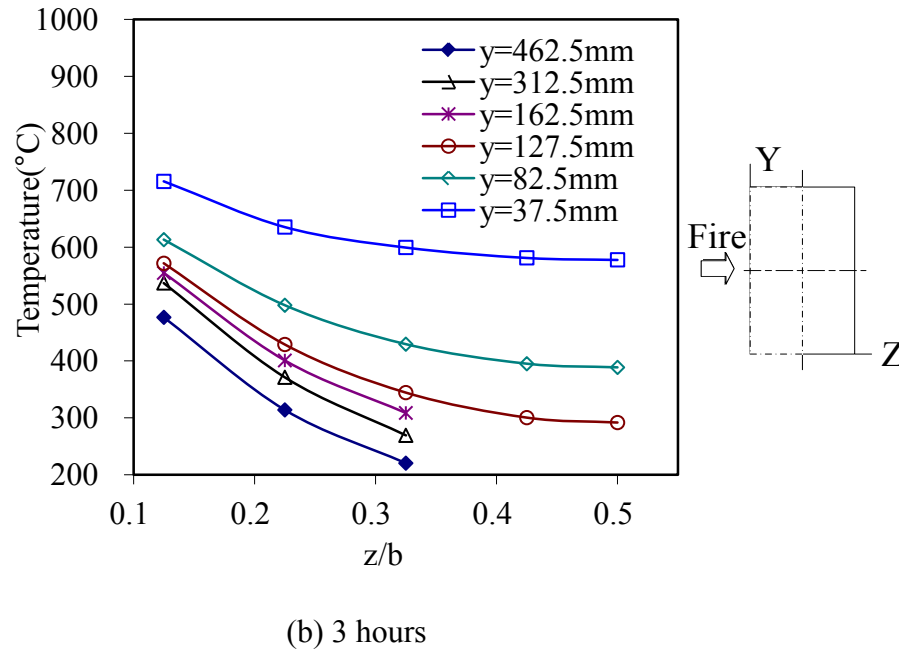


Figure 7.2 Variation of temperature with distance from the side surface of an RC beam at various times (section 300×500mm)

Figure 7.2 (cont'd)



Thermal analysis was carried out on all RC beams listed in Table 7.1. It is found that for beams with width ranging from 200 to 500mm and the depth ranging from 400 to 700mm, cross-sectional temperature is dominated by the distance (depth and width) from the fire exposed surfaces, but barely influenced by z/b and y/h ratio. Based on the influencing area of each fire exposure side, the division criteria for different heat transfer areas in these beams can be given as follows: for the portion of the section where $z/b < 0.5$ and $y/h < 0.5$, 2-D heat transfer occurs, since the resulting temperature is the effect of heat transfer from the bottom and side surfaces. For the portion of the section where $z/b < 0.5$ and $y/h > 0.5$, 1-D heat transfer occurs since the resulting temperature is affected by heat transfer from one side. The section within $z/b > 0.5$, can also be divided into 1-D and 2-D heat transfer area according to the symmetry of geometry and fire-exposure, as shown in Figure 7.3. Similarly, by considering the fire exposure conditions and section properties, the columns, slabs and walls could also be divided into 1-D and 2-D heat transfer areas

for temperature calculation, as shown in Figure 7.3. For concrete members with width (or depth) smaller than 200mm, the above criteria may not be applicable, since the cross-sectional temperatures of these members is influenced by heat transfer from multiple fire exposed surfaces.

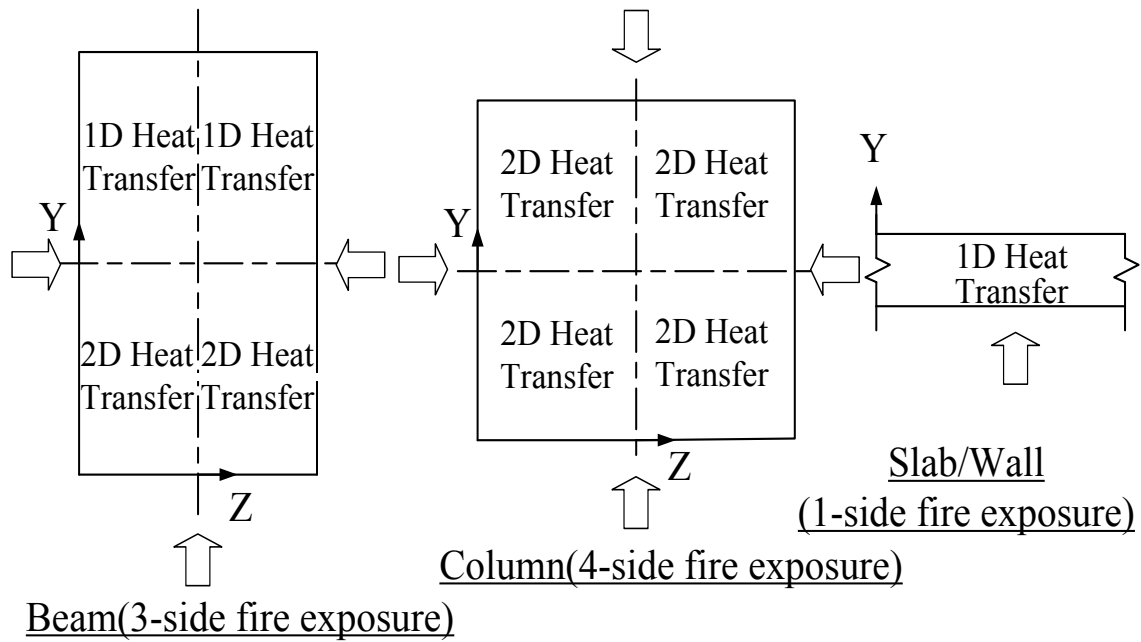


Figure 7.3 Cross section idealization for heat transfer analysis in concrete members exposed to different fire conditions

7.2.1.4 Nonlinear regression analysis

A nonlinear regression analysis on temperature data with corresponding fire-exposed time and cross-sectional locations was carried out using “solver” function in Microsoft Excel (2010). The “solver” function is able to calculate the optimum coefficients to match the original data with a given format of formula and applied “constraint” criteria. The extent of “optimum”, which is the error between the predictions and original data, highly depends on the format of formula and constraint criteria.

Therefore, the basic format of the equations and constraint criteria were to be developed first before undertaking regression analysis.

Based on trial analysis and sensitivity studies, the general format of the temperature equation subjected to regression is deduced by applying similar analogy to the temperature equation proposed by Wickstrom (1986). The equation for temperatures controlled by 1-D heat transfer can be expressed as:

$$T_c = c_1 \cdot \eta_z \cdot (at^n) \quad (7.5)$$

$$\eta_z = a_1 \cdot \ln \frac{t}{z^{1.5}} + a_2 \cdot \sqrt{z} + a_3 \quad (7.6)$$

where, T_z is the temperature resulting from 1-D heat transfer in °C, η_z is the heat transfer factor induced through one fire-exposed surface, c_1 is the coefficients to account for concrete type, t is the fire exposure time in hours, z is the distance from the point in concrete section to fire exposure surface in meters, a_1 , a_2 and a_3 are the coefficients to be traced in the regression analysis. at^n is the temperature under standard fire exposure (Dwaikat and Kodur 2013). For ISO 834 fire, $a = 935$ and $n = 0.168$, and for ASTM E119 fire, $a = 910$ and $n = 0.148$.

For 2-D heat transfer, the temperature equation is obtained by combining the heat from each side of fire exposure:

$$T_c = c_2 \cdot (b_1 \cdot (\eta_z \cdot \eta_y) + b_2 \cdot (\eta_z + \eta_y) + b_3)(at^n) \quad (7.7)$$

where, T_{yz} is the temperature resulting from 2-D heat transfer in °C, c_2 are the coefficients to account for concrete type, η_z and η_y are the heat transfer factors resulting

from y and z side fire exposure, η_y is calculated in the same manner as that of η_z in Eq.

7.2. b_1 , b_2 and b_3 are the coefficients to be traced in the regression analysis. The default value of c_1 and c_2 is 1.0 for normal strength carbonate aggregate concrete. In the regression analysis, the regression on Eq. 7.1 was carried out at first to find the value of a_1 , a_2 and a_3 , and then these obtained values were used to get b_1 , b_2 and b_3 for 2-D heat transfer formula.

In reality, the regression analysis can hardly match all the data points closely. Therefore, it is necessary to fit the data points in the critical range with the smallest discrepancy, and that have reasonable match in other regions using “constraint” criteria. It is well established that the compressive strength of concrete and yield strength of reinforcing steel are not influenced up to 300°C, and that these strengths become negligible after reaching 800°C (ACI 216 2007, Eurocode 2 2004). Therefore, the regression result has to be highly reliable or slightly conservative in temperature-sensitive zone of 300-800°C. Further, the regression results in 20-300°C could be set as a secondary target since the variation in this temperature range does not significantly influence the strength of concrete and steel reinforcement. To achieve this objective, the following constraint criteria were applied in the regression analysis:

- a. When temperature is in 300-800°C range, the average of errors between temperatures by FEA and predicted temperatures by regression equations should be controlled within 10%.
- b. For temperature higher than 800°C, predicted temperature using regression equations should not be smaller than those from FEA.

- c. For temperature in 100-300°C, the average of errors between temperatures by FEA and predicted temperatures by regression equations should be controlled within 15%.
- d. For temperature from FEA smaller than 100°C, predicted temperatures by regression equations should not be lower than those from FEA by more than 50°C. When predicted temperature is lower than 20°C, it is taken as 20°C.

With the above developed equations and constrains, a regression analysis was conducted for 1-D and 2-D heat transfer equations for each types of concrete. The final formulae used for calculating temperature at a given point in an RC member are obtained as follows.

$$\text{For 1-D heat transfer: } T_c = c_1 \cdot \eta_z \cdot (at^n) \quad (7.8)$$

$$\text{where, } \eta_z = 0.155 \ln \frac{t}{z^{1.5}} - 0.348\sqrt{z} - 0.371 \quad (7.9)$$

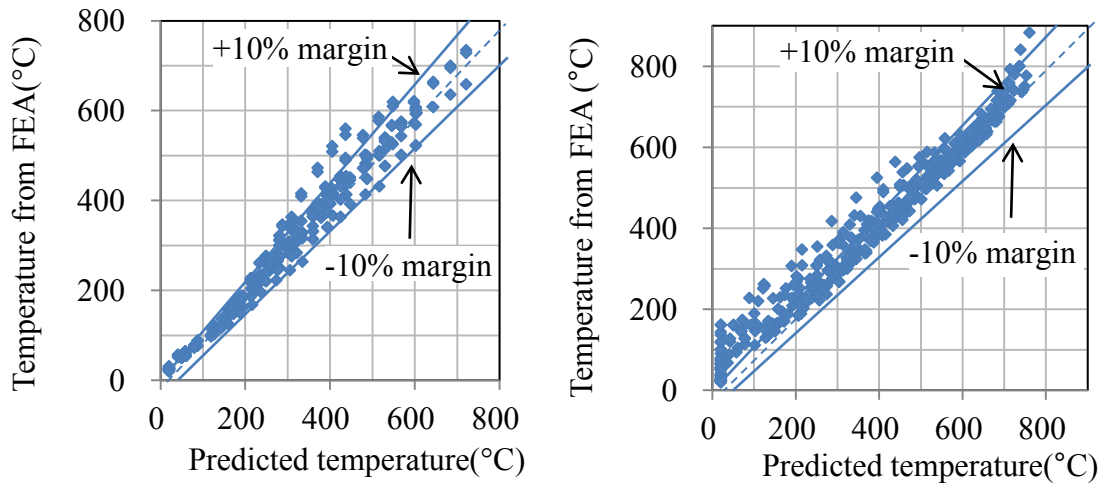
$$\text{For 2-D heat transfer: } T_c = c_2 \cdot (-1.481 \cdot (\eta_z \cdot \eta_y) + 0.985 \cdot (\eta_z + \eta_y) + 0.017)(at^n) \quad (7.10)$$

where c_1 are 1.0, 1.01, 1.12 and 1.12 for NSC-CA, HSC-CA, NSC-SA and HSC-SA, respectively; c_2 are 1.0, 1.06, 1.12 and 1.20 for NSC-CA, HSC-CA, NSC-SA and HSC-SA, respectively.

7.2.1.5 Regression analysis results

The temperature predictions from proposed equations (Eqns. 7.8 and 7.10) are compared with the regression data obtained from detailed finite element analysis. These comparisons are plotted in Figure 7.4 for 4 RC groups of beams made of NSC-CA, HSC-CA, NSC-SA and HSC-SA. In these figures, a point below “-10% margin” line indicate

that the predicted temperature from equations is to be higher than that obtained in FEA by more than 10%. If a point lies above “+10% margin” line, the predicted temperature from equations is smaller than that obtained in FEA by more than 10%. It can be seen that for all four concrete types, most data points lie within $\pm 10\%$ margin zone, especially for temperatures higher than 300°C . Therefore, the proposed equations are capable of predicting cross-sectional temperatures of RC members exposed to standard fire to a good degree. It is noted that there are a few points in the zone above “+10% margin” line, indicating unconservative temperature predictions. These points correspond to sections with smaller width (200×500 mm) and this inaccuracy could be attributed to the fact those cross-sectional temperatures result from heat transfer from three fire-exposed surfaces due to the smaller width, while the proposed formulas account for heat transfer from one or two fire-exposed sides, which is true in most practical situations.

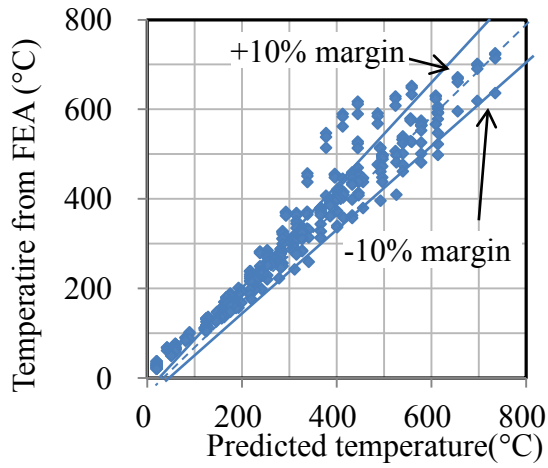


(a) 1-D heat transfer (NSC-CA)

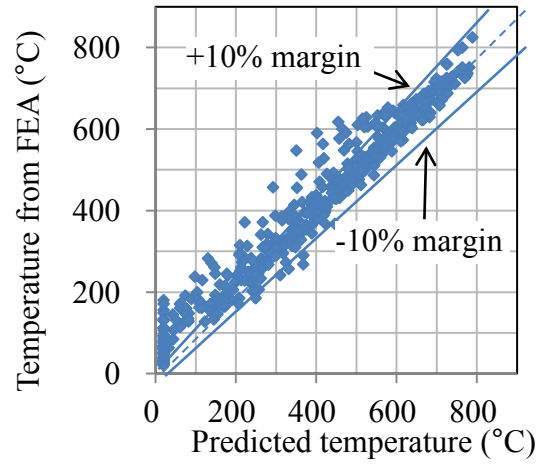
(b) 2-D heat transfer (NSC-CA)

Figure 7.4 Comparison of predicted temperatures from the proposed equations with those from FEA

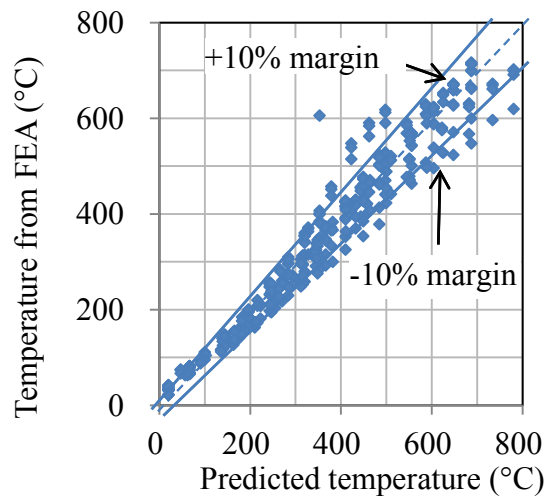
Figure 7. 4 (cont'd)



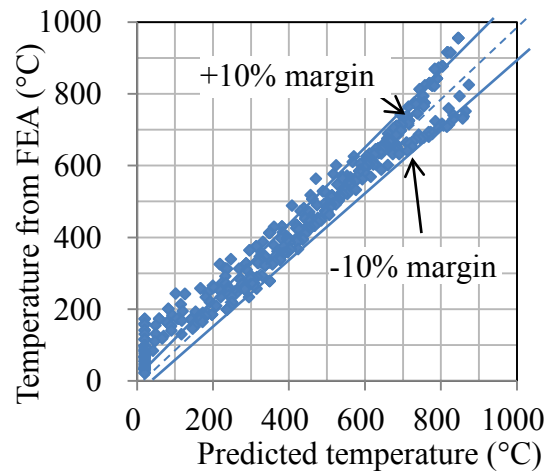
(c) 1-D heat transfer (HSC-CA)



(d) 2-D heat transfer (HSC-CA)

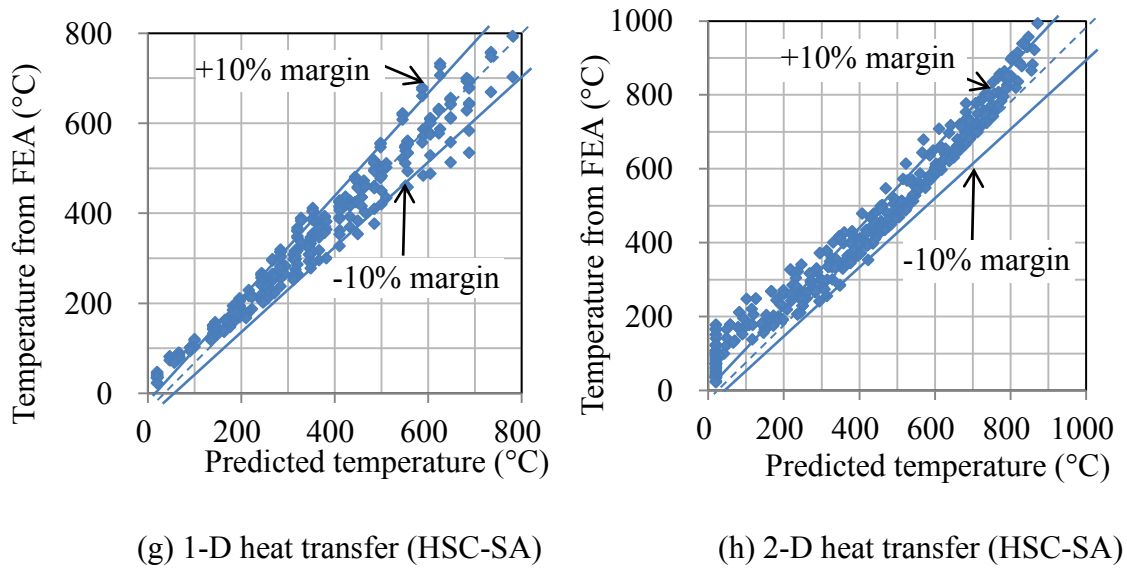


(e) 1-D heat transfer (NSC-SA)



(f) 2-D heat transfer (NSC-SA)

Figure 7. 4 (cont'd)



7.2.1.6 Verification of temperature equations using test results

The validity of the proposed equations is established by comparing the predicted temperatures (using Eqns. 7.8 and 7.10) with the measured temperatures from fire tests on RC beams, columns and slabs. In total, 5 beams, 5 columns and 1 slab tested in the literature (Kodur et al. 2006, Raut and Kodur 2011, Lin 1981, Dwaikat and Kodur 2009, Kodur and Bisby 2005, Dotreppe and Franssenm 1985, Kodur et al. 2004), were selected for this validation. The properties of selected RC members cover different concrete types (NSC-CA, HSC-CA, NSC-SA and HSC-SA) and width-depth ratios (ranging from 0.33 to 1.0). The tested beams, columns and slab were subjected to three-side, four-side and one-side fire exposure, respectively. The details of tested members used for validation are shown in Table 7.2.

The predicted temperatures using Eqns. 7.8 and 7.10, for NSC-CA, are compared with measured values in fire tests in Figure 7.5. It can be seen that the predicted rebar

temperatures are generally in good agreement with the measured values in the beams and columns. In the initial stage, the predicted rebar temperatures are slightly higher than the measured ones in fire tests, due to relatively larger discrepancy in 20-300°C range in the regression analysis. In Figures 7.5e and 7.5f, the temperatures in the slabs, calculated using 1-D heat transfer equation, are also compared to measured temperatures in the test. It can be seen the predictions agree with the test results throughout the slab thickness, which demonstrates that the proposed 1-D heat transfer equation can well handle the slab temperature problem.

To check the validity of the proposed equations over a wide range of scenarios, the predicted temperatures are compared against the measured temperatures in fire test on RC members made of different types of concrete (HSC-CA, NSC-SA and HSC-SA). The comparisons are plotted in Figures 7.6-7.8. Since the proposed equations utilizes factors (c_1 and c_2) to account for different aggregate types and concrete types, the cross-sectional temperatures predicted by those equations have a close match with the measured temperatures. Also, by using 1-D or 2-D equations, the temperatures in both rebar and concrete at various depths could be predicted with a good accuracy. Thus, Eqns. 7.8 and 7.10 are suited to predict cross-sectional temperatures in structural members made of different types of concrete.

Table 7.2 Sections of RC members used in validation of temperature equations

Group	Concrete Strength	Aggregate	Cross-sectional dimensions			
			Finite element analysis		Test	
			Dimension (mm)	b/h	Dimension (mm)	b/h
I	NSC	Carbonate	B400×700	0.57	Column 406×406 (Kodur et al. 2006)	1.0
					Column 203×203 (Raut and Kodur 2011)	1.0
			B250×450	0.56	Beam 229×533 (Lin 1981)	0.43
					Beam 254×408 (Dwaikat and Kodur 2009)	0.62
					Slab 152 (Kodur and Bisby 2005)	--
II	HSC	Carbonate	B400×700	0.57	Column 203×203 (Raut and Kodur 2011)	1.0
			B250×450	0.56	Beam 254×408 (Dwaikat and Kodur 2009)	0.62
III	NSC	Siliceous	B400×700	0.57	Beam 200×600 (Dotreppe and Franssenm 1985)	0.33
			B250×450	0.56	Column 305×305 (Kodur et al. 2003)	1.0
IV	HSC	Siliceous	B400×700	0.57	Column 305×305 (Kodur et al. 2003)	1.0
			B250×450	0.56		

In Figures 7.5-7.8, the temperature predictions using Wickstrom's equation [1986] are also plotted. It can be seen that at the location of rebars, the predicted temperatures using Wickstrom's equation are much higher than the measured values in fire tests, while at mid-depth of concrete, Wickstrom's equation predicts lower temperatures than the measured ones. One possible reason for this discrepancy is due to the fact that Wickstrom's equation does not account for the variation resulting from thermal

properties of different concrete types (NSC-CA, HSC-CA, NSC-SA and HSC-SA). Such large discrepancy on cross-sectional temperatures could affect fire resistance calculations of an RC member. Therefore, the proposed equations can provide better estimate of temperatures, and thus are more suited for fire resistance evaluation.

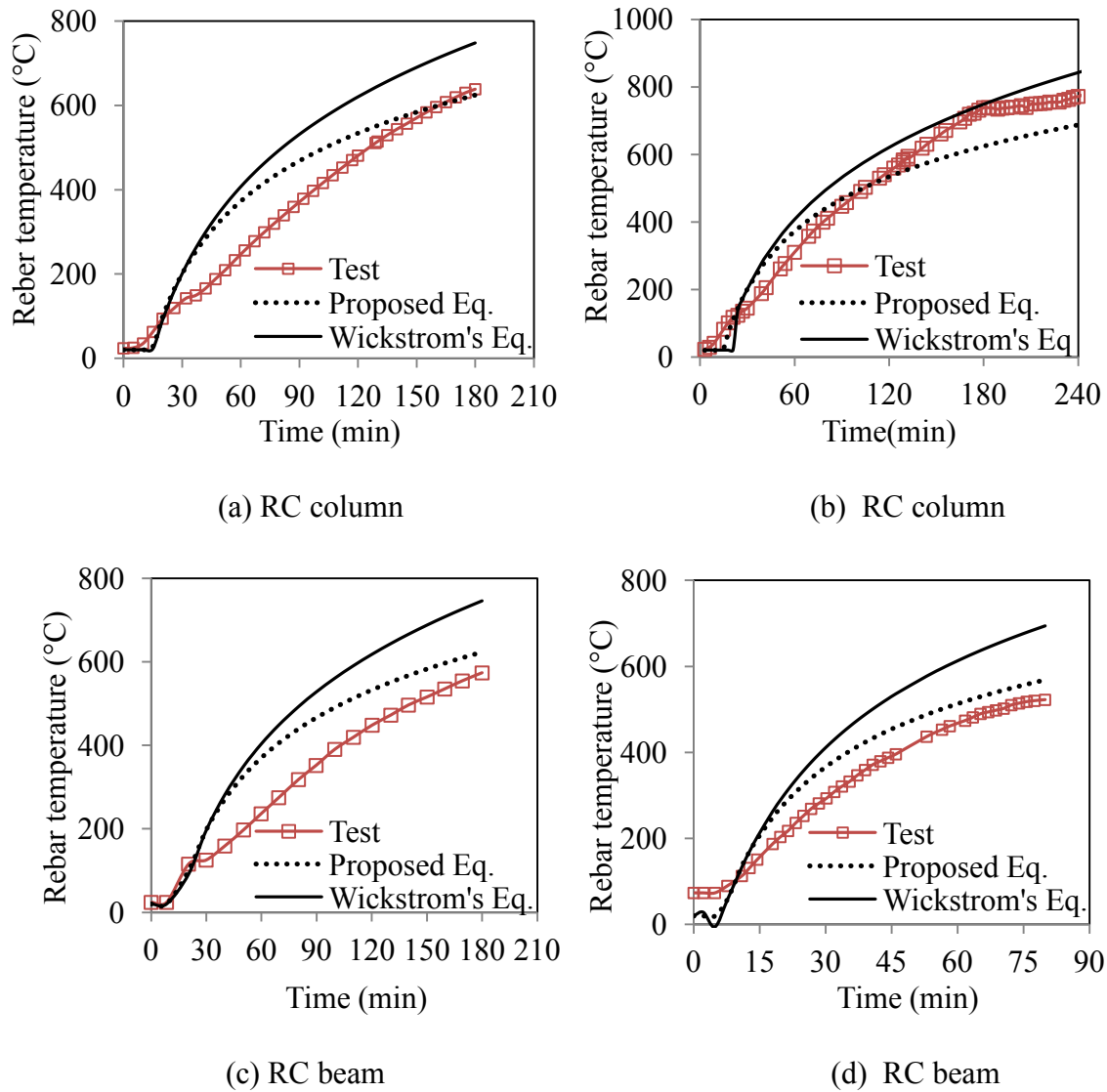
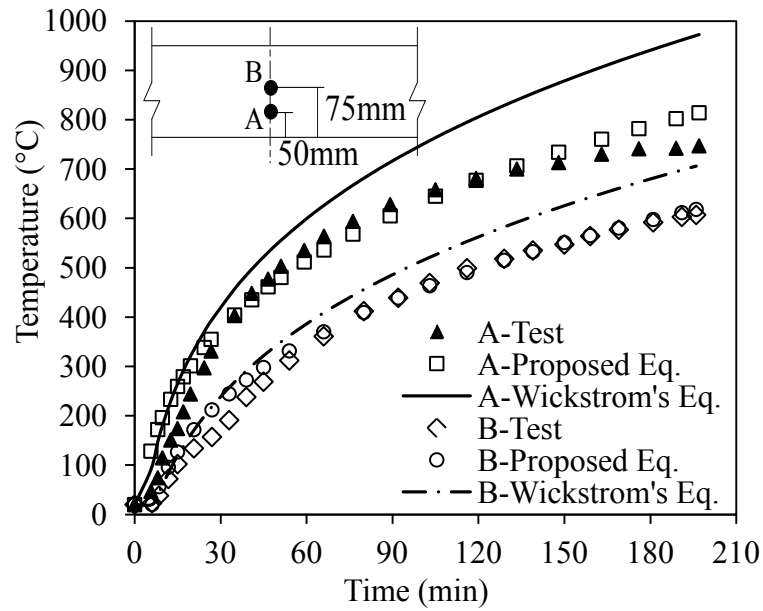
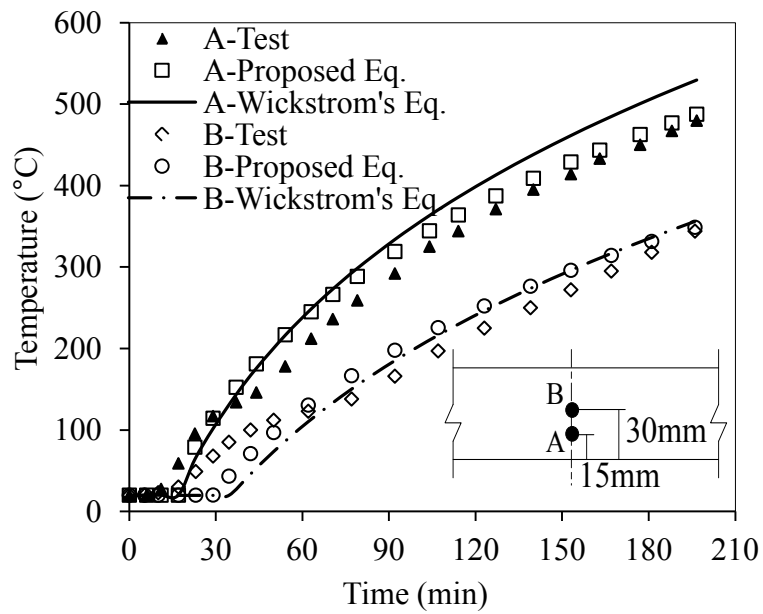


Figure 7.5 Validation of the proposed approach by comparing predicted and measured temperatures for NSC-CA members

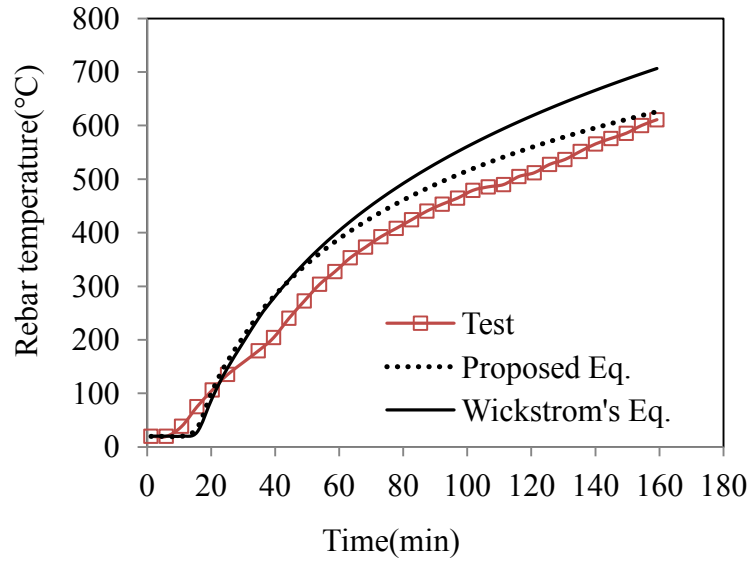
Figure 7.5 (cont'd)



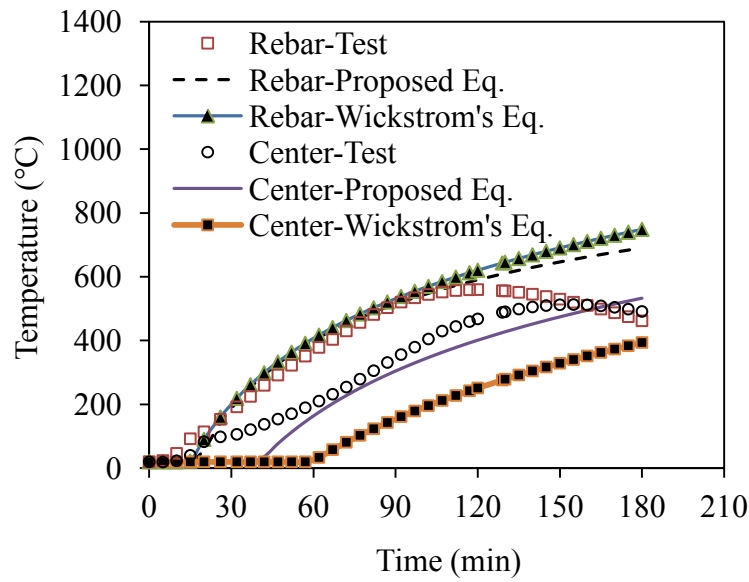
(e) RC slab



(f) RC slab

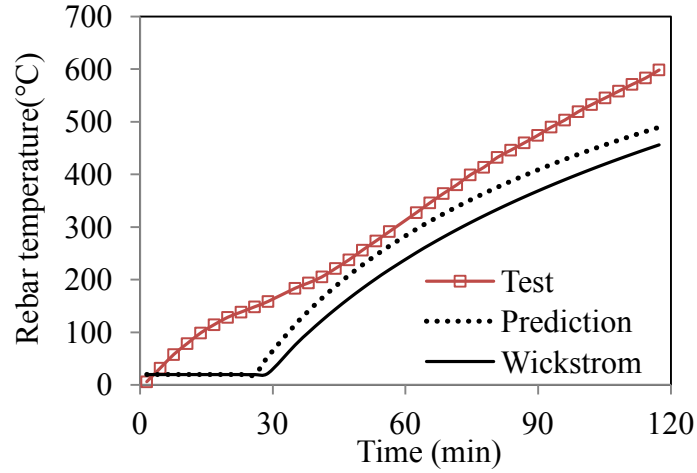


(a) RC beam

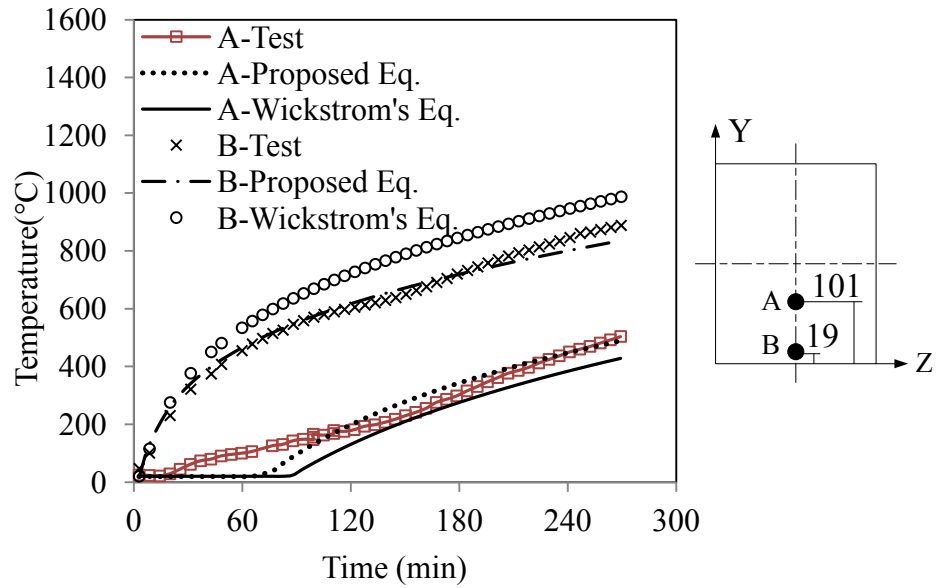


(b) RC column

Figure 7.6 Validation of the proposed approach by comparing predicted and measured temperatures for HSC-CA members



(a) RC beam



(b) RC column

Figure 7.7 Validation of the proposed approach by comparing predicted and measured temperatures for NSC-SA members

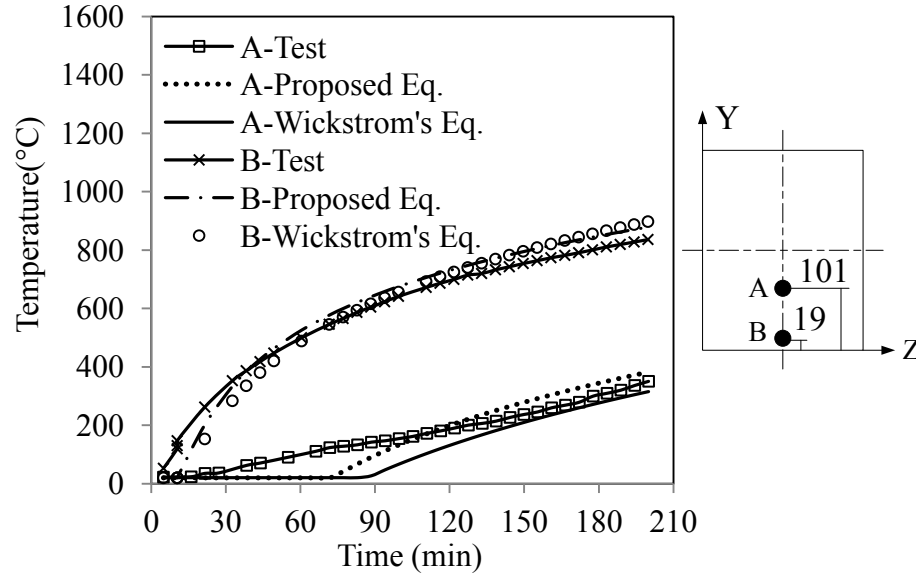


Figure 7.8 Validation of the proposed approach by comparing predicted and measured temperatures for HSC-SA members

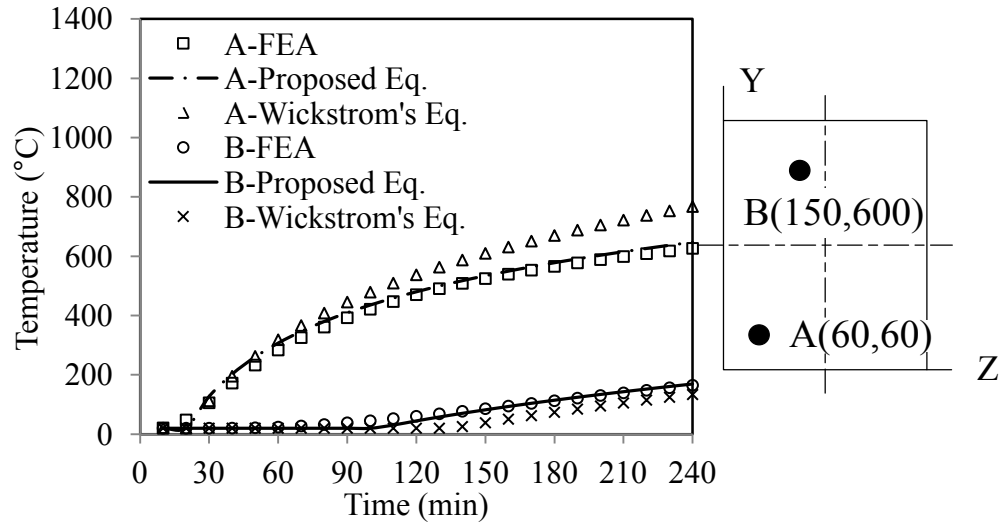
7.2.1.7 Verification of temperature equations using FEA Results

The above developed simplified equations for temperature predictions are further validated by comparing temperature predictions with those obtained from FEA. To demonstrate the applicability of the proposed equations to a wide range of situations, the selected dimensions of concrete members for FEA are different from those of the tested concrete members used in Section 7.2.1.2, since test specimens are generally of smaller dimensions. As shown in Table 7.2, the cross section of beams used for validation are 400×700 mm and 250×450 mm, and the high-temperature material properties of these two beams are varied to take into account for NSC-CA, HSC-CA, NSC-SA and HSC-SA. Two points in each beam, located in 1-D and 2-D heat transfer area, respectively, are selected for comparisons between predictions by proposed equations and FEA.

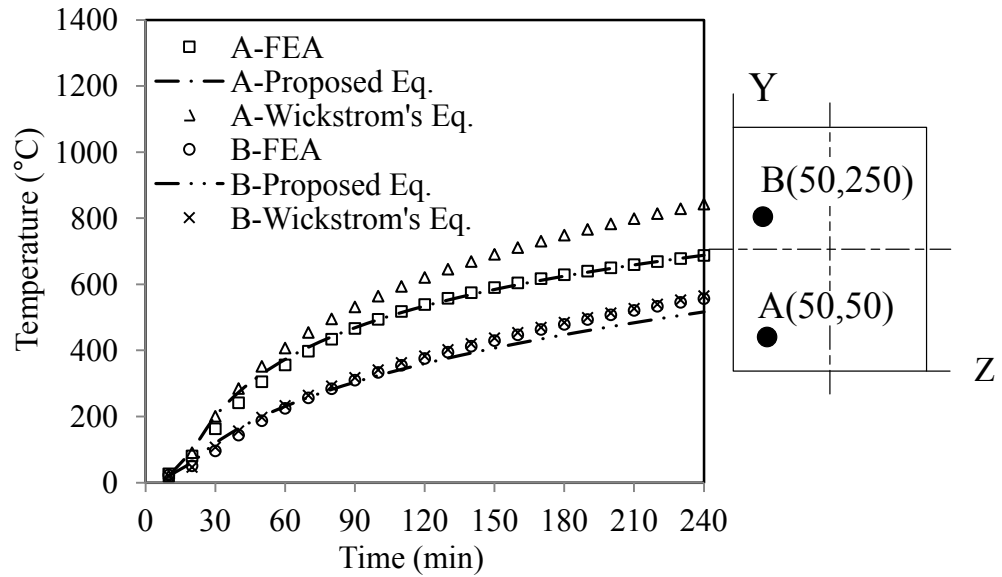
The comparison of temperature predictions using proposed equations and FEA is shown in Figures 7.9-7.12. It can be seen that for all beams under consideration, the

proposed equations are able to accurately predict cross-sectional temperatures in the entire range of fire exposure. Since the two points in each beam are using 1-D and 2-D heat transfer equations, respectively, the reasonability of cross-sectional division are also validated. Comparing to the test results validation, the predictions by the proposed equation have better agreement with FEA results. This is probably due to the fact that the proposed equation is derived based on the results from finite element analysis.

The predicted temperatures using Wickstrom's equation are also compared to FEA results in Figures 7.9-7.12. Wickstrom's equation gives the same temperature predictions for the beams with different concrete types as can be seen in the figures, and this is due to the fact that Wickstrom's equations does not account for specific concrete type. The predicted temperatures from Wickstrom's equation are in better agreement for siliceous aggregate concrete members, but for carbonate aggregate concrete the predictions are too conservative.

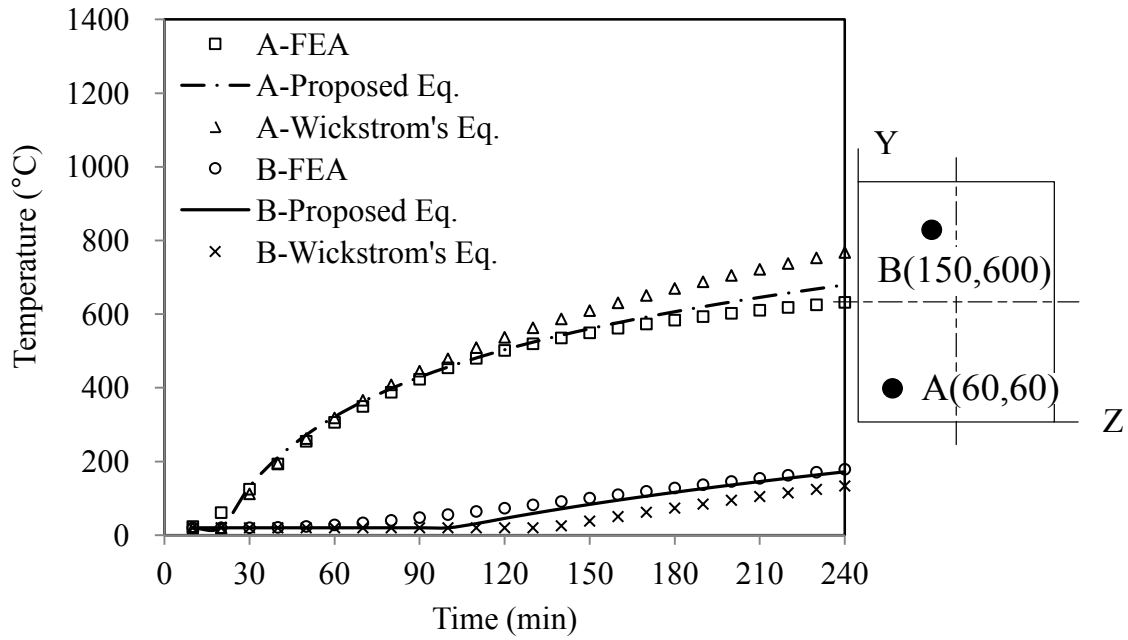


(a) Beam 400×700 mm

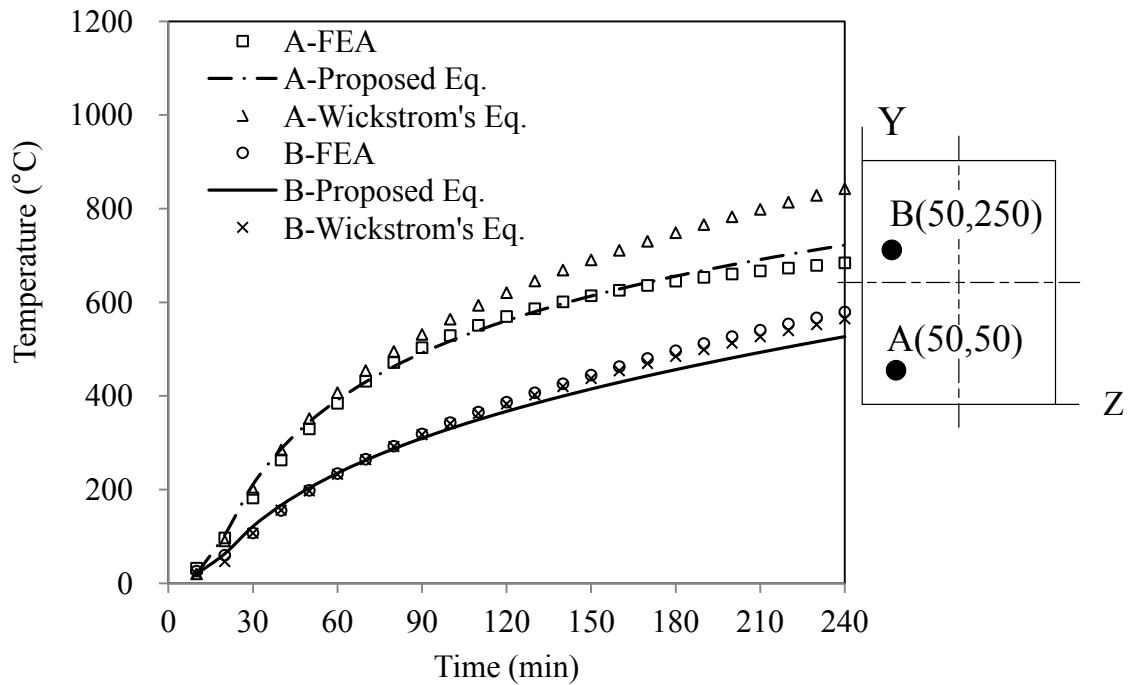


(b) Beam 250×450 mm

Figure 7.9 Validation of the proposed approach by comparing predicted temperatures with FEA results for NSC-CA members

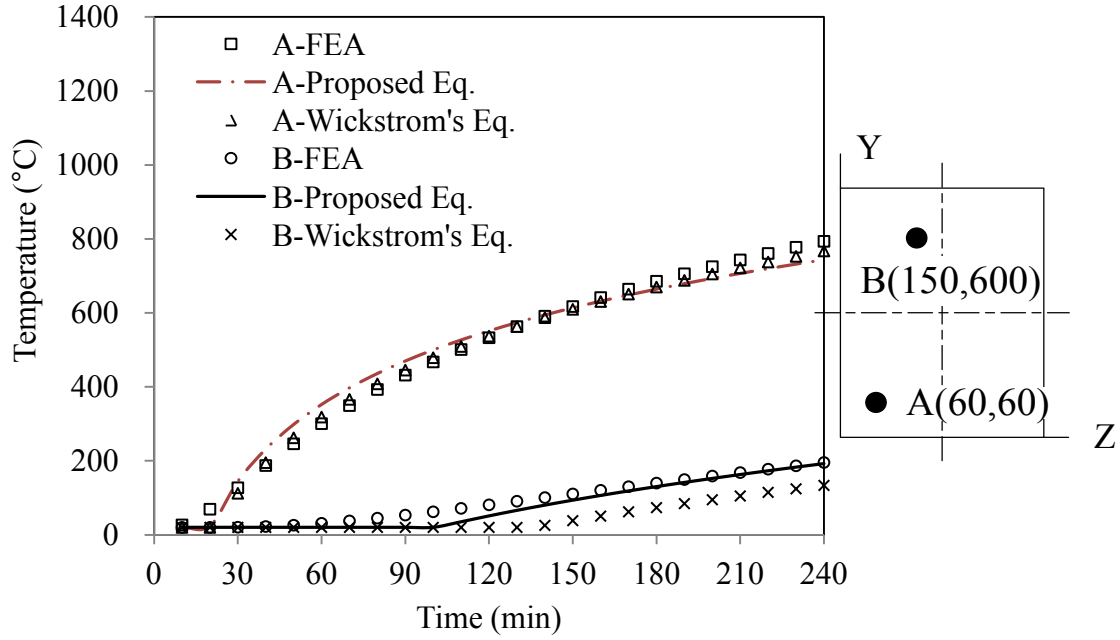


(a) Beam 400×700 m

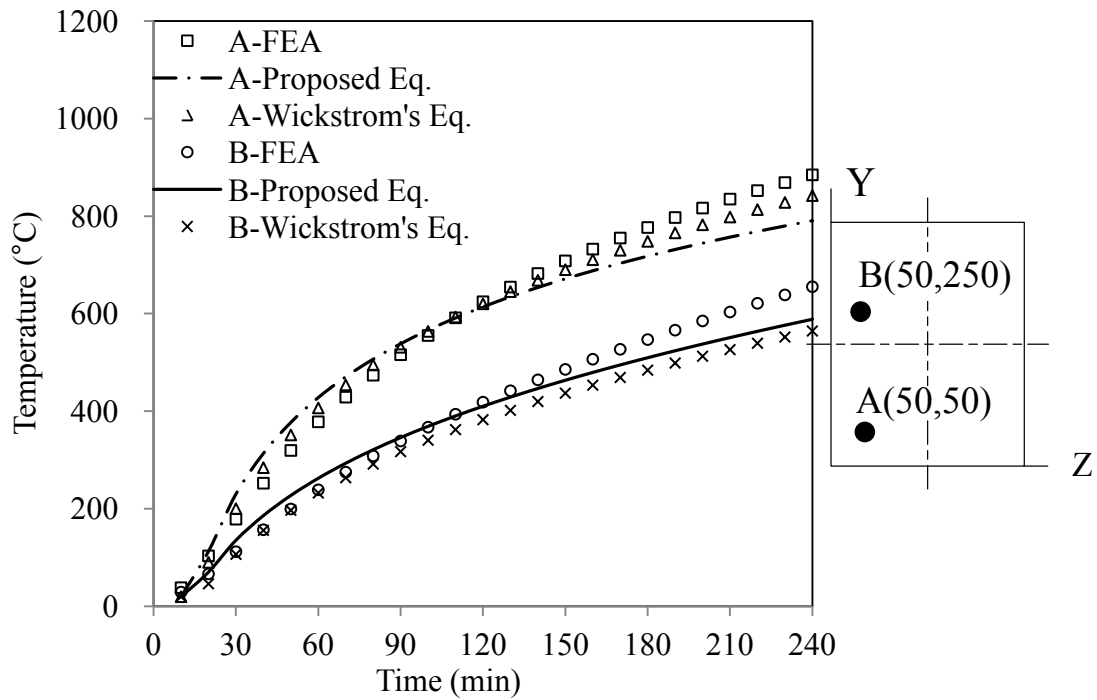


(b) Beam 250×450 mm

Figure 7.10 Validation of the proposed approach by comparing predicted temperatures with FEA results for HSC-CA members

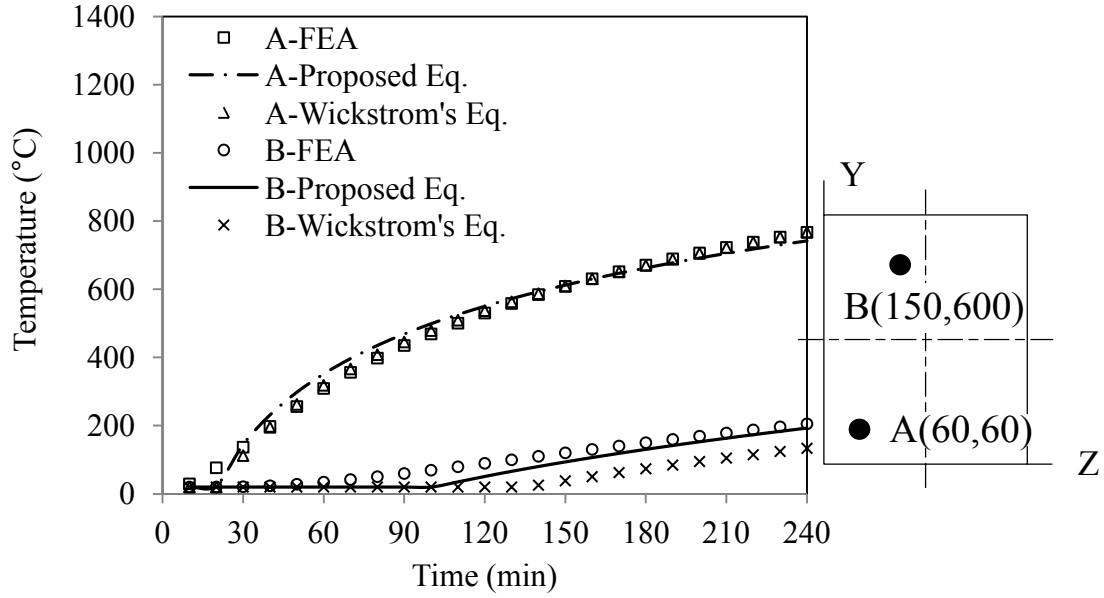


(a) Beam 400×700 mm

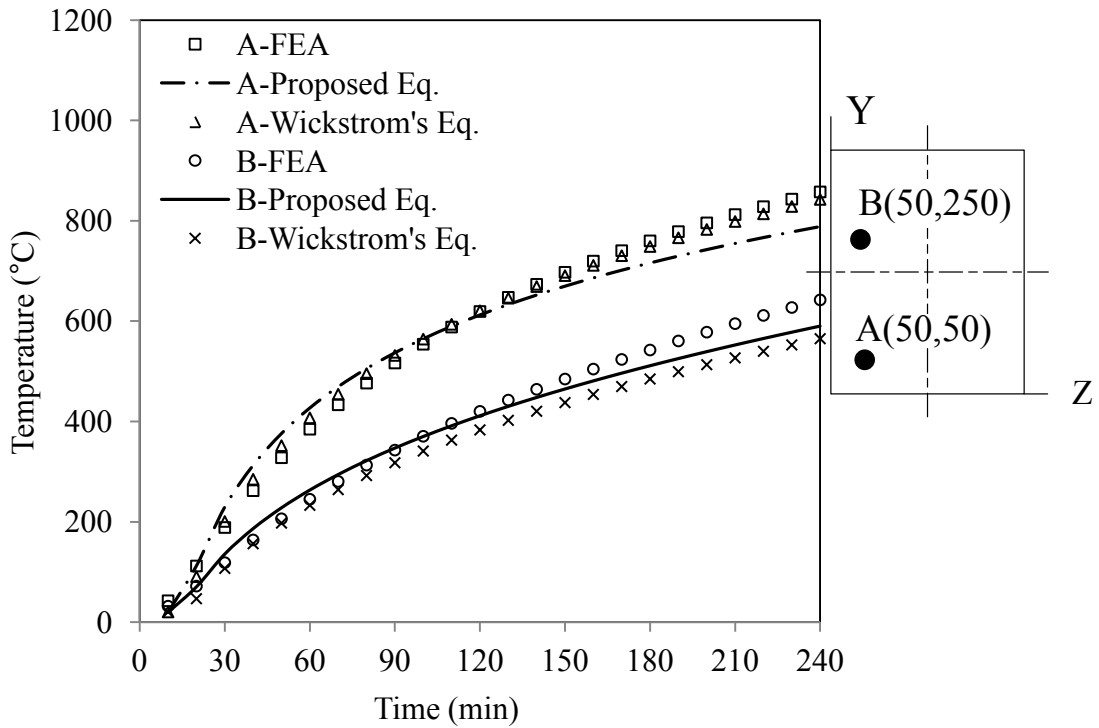


(b) Beam 250×450 mm

Figure 7.11 Validation of the proposed approach by comparing predicted temperatures with FEA results for NSC-SA members



(a) Beam 400×700 mm



(b) Beam 250×450 mm

Figure 7.12 Validation of the proposed approach by comparing predicted temperatures with FEA results for HSC-SA members

7.2.2 An approach for predicting temperatures in an insulated RC member

The above simplified approach for evaluating temperature in RC members is extended to cover the temperature predictions in an insulated RC member. Unlike a conventional RC member, temperature profiles in an insulated RC member are influenced by properties of both concrete and fire insulation. Since there is a large difference between thermal properties of concrete and insulation, also the heat flux on the boundary of concrete and insulation is different, predicting temperature in an insulated concrete member becomes a more complicate problem. So far there is no simplified approach for evaluating temperature in an insulated concrete member.

This section provides development of simplified approach for evaluating temperature profiles in an insulated RC member. A simple expression for converting insulation layer to equivalent concrete layer is firstly derived. Then regression analysis on a large amount of temperature data, collected using detailed thermal analysis on RC members, is conducted to obtain the final temperature equations. Since FRP or steel reinforcement does not significantly affect the temperature rise in an insulated RC member, the proposed equations are suitable for evaluating temperatures in RC members with various configurations, such as NSM or external FRP strengthened RC members, concrete members with internal FRP rebar, etc. Detailed derivation and regression procedure are presented as follows.

7.2.2.1 Converting fire insulation layer to equivalent concrete layer

For an FRP strengthened RC member (with or without fire insulation), temperatures at critical locations such as FRP and steel rebar, dominate the behavior of

strengthened members under fire conditions. Thus temperatures in concrete, steel and FRP need to be known, while temperature in insulation layer is not of much interest. It is well established that thermal properties of concrete and insulation primarily influence temperature rise within a concrete member. Thermal properties of steel rebar and FRP (laminates, strip or rod) does not significantly affect temperature rise in RC member due to their small cross sectional areas as compared to concrete section (Lie and Irwin 1993). Thus, if the insulation layer on concrete member is converted to equivalent concrete layer, temperature profile in an insulated concrete section can be evaluated using the same temperature equation as that for uninsulated RC sections (Eqns. 7.8 and 7.10).

Figure 7.13 shows a typical RC beam protected by a U-shaped fire insulation. The thickness of insulation on sides and bottom of the beam are z_i and y_i respectively. Assuming that fire insulation layer can be replaced by an equivalent concrete layer with the thickness of z_{ec} and y_{ec} on sides and bottom respectively, and temperature profiles remain the same within beam cross section after this alternation. Based on the heat transfer principles, the following equations can be obtained within the insulation or its equivalent concrete layer.

$$\frac{k_i}{(\rho c)_i} \nabla^2 T_i = \frac{\partial T_i}{\partial t} - \frac{Q}{(\rho c)_i} \quad (7.11)$$

$$\frac{k_c}{(\rho c)_c} \nabla^2 T_{ec} = \frac{\partial T_{ec}}{\partial t} - \frac{Q}{(\rho c)_c} \quad (7.12)$$

where k_i and $(\rho c)_i$ are thermal conductivity and heat capacity of insulation respectively, and k_c and $(\rho c)_c$ are thermal conductivity and heat capacity of concrete respectively; Q is

the heat source; T_i and T_{ec} are the temperatures in insulation and equivalent concrete respectively; t is the time; and ∇ is the second order derivative of temperature (T_i or T_{ec}) with respect to the distance to fire exposed surface (z or y).

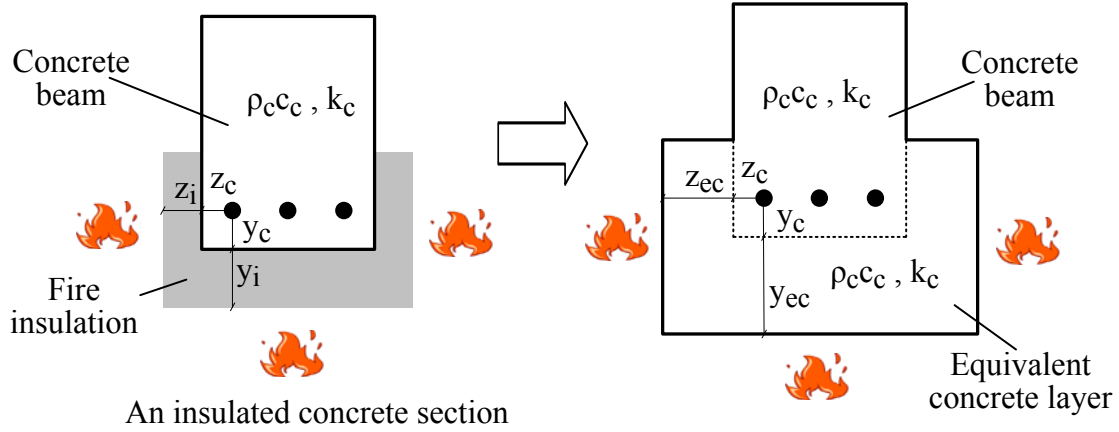


Figure 7.13 Illustration of the equivalent concrete depth method

Eqns. 7.11 and 7.12 represent temperature distributions within insulation layer or equivalent concrete layer. At the boundary of the insulation layer ($z = z_i$), temperatures remain same before and after alternation (see Figure 7.13). Thus Eq. 7.13 can be obtained.

$$\left. \frac{k_c}{(\rho c)_c} \frac{\partial^2 T_{ec}}{\partial z^2} \right|_{z=z_{ec}} \approx \left. \frac{k_i}{(\rho c)_i} \frac{\partial^2 T_i}{\partial z^2} \right|_{z=z_i} \quad (7.13)$$

where z is the distance from fire exposed surface. Since 1-D heat transfer dominates temperature distribution within insulation or concrete layer due to their small thickness, temperature equation resulting from 1D heat transfer (Eq. 7.8) can be used. T_{ec} in Eq. 7.13 can be expressed as:

$$T_{ec} = (0.155 \ln \frac{t}{z^{1.5}} - 0.348 \sqrt{z} - 0.371) \cdot (at^n) \approx (0.155 \ln \frac{t}{z^{1.5}} - 0.371) \cdot (at^n) \quad (7.14)$$

where at^n is the standard fire temperature (Dwaikat and Kodur 2013). For ISO 834 fire, $a = 935$ and $n = 0.168$, and for ASTM E119 fire, $a = 910$ and $n = 0.148$. Assume that temperature distribution within insulation is similar to that within concrete, and then T_{ec} in Eq. 7.13 can be expressed as

$$T_i \approx (a_1 \ln \frac{t}{z^{1.5}} - a_2) \cdot (at^n) \quad (7.15)$$

where a_1 and a_2 are the coefficients describing temperature distribution within insulation layer. Importing Eqns. 7.14 and 7.15 into Eq. 7.13, the following relation between the thickness of insulation layer (z_i) and concrete layer (z_{ec}) can be obtained

$$\frac{z_{ec}}{z_i} = \eta \sqrt{\frac{0.155}{a_1} \frac{k_c}{(\rho c)_c} \frac{(\rho c)_i}{k_i}} \approx \lambda \eta \sqrt{\frac{k_c}{(\rho c)_c} \frac{(\rho c)_i}{k_i}} \quad (7.16)$$

where λ and η are the coefficients to be determined. Notice that Eq. 7.16 describes an approximate relation between insulation thickness (z_i) and the equivalent concrete thickness (z_{ec}). Since thermal properties of insulation and concrete vary with temperature (or fire exposure time), λ is assumed to be as a function of time (t) in hours, accounting for the influence of fire exposure time. Finally, the following expression is arrived:

$$\frac{z_{ec}}{z_i} = \alpha \sqrt{t} \beta \sqrt{\frac{k_c}{(\rho c)_c} \frac{(\rho c)_i}{k_i}} \quad (7.17)$$

where α and β are the coefficients to be determined. Once the values of α and β are obtained, the insulation thickness can be converted to an equivalent concrete thickness. For example, Figure 7.13 shows a point (Point A) located in an insulated RC beam. The distances from Point A to side and bottom surface of beam are z_c and y_c respectively, and

insulation thickness on side and bottom surface of beam are z_i and y_i respectively. Using the equivalent concrete depth method, the distance from Point A to fire exposed surface can be expressed as:

$$z_c' = z_c + z_{ec} = z_c + z_i \sqrt[\alpha]{t} \beta \sqrt{\frac{k_c}{(\rho c)_c} \frac{(\rho c)_i}{k_i}} \quad (7.18)$$

$$y_c' = y_c + y_{ec} = y_c + y_i \sqrt[\alpha]{t} \beta \sqrt{\frac{k_c}{(\rho c)_c} \frac{(\rho c)_i}{k_i}} \quad (7.19)$$

Then z_c' and y_c' can be applied into equations used for predicting temperatures in an RC member (Eqns. 7.8 and 7.10) to obtain temperature profiles in an insulated RC member, as shown below.

$$\text{For 1-D heat transfer: } T_c = c_1 \cdot \eta_z \cdot (at^n) \quad (7.20)$$

$$\text{where, } \eta_z = 0.155 \ln \frac{t}{(z')^{1.5}} - 0.348 \sqrt{z'} - 0.371 \quad (7.21)$$

$$\text{For 2-D heat transfer: } T_c = c_2 \cdot (-1.481 \cdot (\eta_z \cdot \eta_y) + 0.985 \cdot (\eta_z + \eta_y) + 0.017)(at^n) \quad (7.22)$$

Where t is the fire exposure time in hours, z' is the distance from the point in concrete section to fire exposed surface using the equivalent concrete depth method (Eqns. 7.18 and 7.19), the meaning of other symbols is illustrated in Section 7.2.1.3.

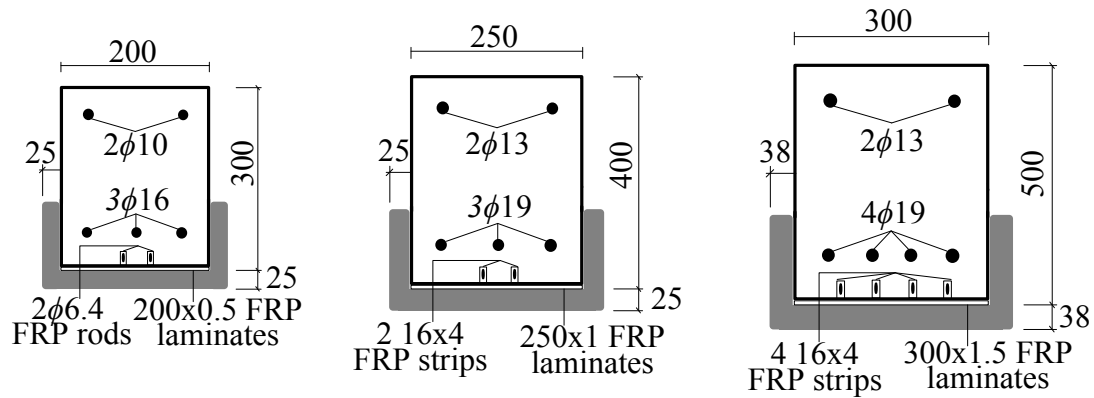
Eqns. 7.18-7.22 provide a simplified approach for evaluating temperatures within an insulated RC beam, including the cases of FRP-strengthened members with insulation. The only unknowns to be determined are α and β in Eqns. 7.18 and 7.19. These two unknowns are obtained through nonlinear regression analysis on a large database generated using finite element analysis (FEA).

7.2.2.2 Regression analysis

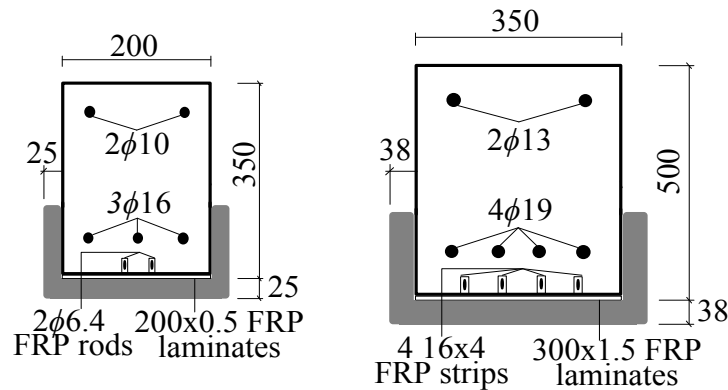
A regression analysis similar to that in Section 7.2.1 was carried out to obtain empirical equations for predicting temperatures in an insulated RC beam. To generate temperature data for regression analysis, three representative RC beams with two types of insulation were analyzed using finite element program described in Chapter 5. The characteristics of these RC beams were varied over a wide range and are shown in Table 7.3 and Figure 7.14. The width of beam section was varied from 200 to 300mm, while the depth was varied from 300 to 500mm. Two types of fire insulation, Aestuver and Tyfo VG insulations, are assumed to be applied on the beams. The variation of thermal properties of fire insulation with temperatures follows previously reported values (Bisby 2003), and they are summarized in Appendix A. Five different points within beam section were selected in each beam, and they are typical locations of steel and FRP reinforcement, as shown in Figure 7.14. In the finite element analysis, each beam was subjected to ASTM E119 fire exposure from three sides for 4 hours, and temperature data at each time interval of 0.5 hours was output for regression analysis. In total, 240 temperature data points with corresponding time and location information ($3 \times 2 \times 5 \times 8$) were generated for regression analysis.

Table 7.3 Characteristics of insulated RC beams used in the regression analysis

Case No.	Section	Fire insulation	Point	Fire exposure time
1	200×300mm 250×400mm 300×500mm	AESTUVER $k = 0.185 W/m-K$ $(\rho c) = 650 kJ/m^3-K$	Corner steel rebar	0.5h, 1h, 1.5h, 2h, 2.5h, 3h, 3.5h, 4h
			Middle steel rebar	
			NSM FRP strip	
			1/4 length of FRP laminate	
			Middle of FRP laminate	
2	200×300mm 250×400mm 300×500mm	Tyfo VG $k = 0.116 W/m-K$ $(\rho c) = 413 kJ/m^3-K$	Corner steel rebar	0.5h, 1h, 1.5h, 2h, 2.5h, 3h, 3.5h, 4h
			Middle steel rebar	
			NSM FRP strip	
			1/4 length of FRP laminate	
			Middle of FRP laminate	



(a) Beams analyzed for regression analysis



(b) Beams analyzed for validation of temperature equations

Figure 7.14 FRP strengthened RC beams used in FEA for regression and validation

(Units: mm)

A regression analysis was performed using “Solver” function in Excel (2010) to develop an expression for converting insulation to an equivalent concrete layer (Eq. 7.17). The “solver” function is able to calculate the optimum coefficients to match the original data with a given format of formula and applied “constraint” criteria. To ensure better accuracy of the final equation, the errors between predicted temperatures using formula and temperatures obtained from FEA were controlled within 10%, or 15% conservative. α and β in Eqns. 7.17 are the coefficients to be determined in the regression analysis.

Then a regression analysis was carried out so as to achieve minimum error between predicted temperature (using Eqns. 7.18-7.22) and temperature obtained from FEA. Based on the regression analysis results, α and β are determined to be 4.5 and 1.75 respectively, and thus the relation between insulation and its equivalent concrete is

$$z_{ec} = z_i^{4.5} \sqrt[1.75]{\frac{k_c}{(\rho c)_c} \frac{(\rho c)_i}{k_i}} \quad (7.23)$$

The temperature predictions using equivalent concrete method (Eqns. 7.18 to 7.22) are compared with temperature data generated from FEA. These comparisons are plotted in Figures 7.15 - 7.17 for three insulated RC beams (two types of insulation). In these figures, a point below “-10% margin” line indicate that the predicted temperature is to be higher than that obtained in FEA by more than 10%. If a point lies above “+10% margin” line, the predicted temperature is smaller than that obtained in FEA by more than 10%. It can be seen that for three insulated RC beams, most data points lie within $\pm 10\%$ margin zone. Therefore, the proposed equivalent concrete method is capable of predicting temperatures in insulated RC beams exposed to standard fire to a good degree. It is noted that there are a few points outside “ $\pm 10\%$ margin” line, indicating the errors are larger

than 10%. However, most of these points are below “-10% margin” line, and they provide conservative predictions of steel and FRP rebar temperatures.

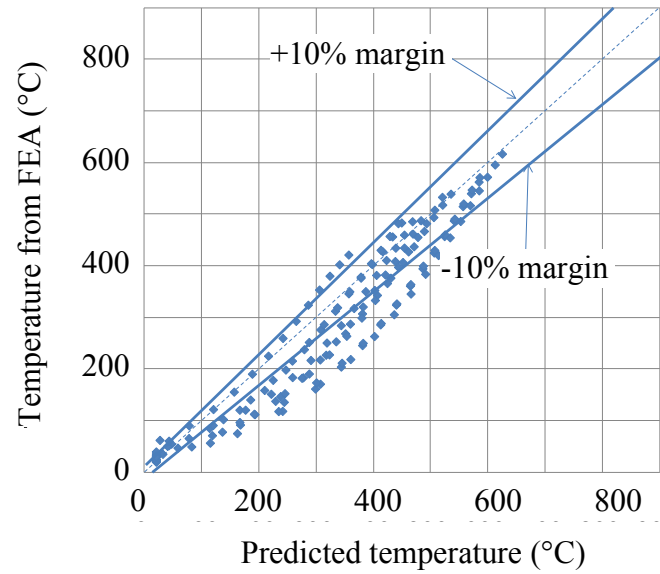


Figure 7.15 Comparison of predicted temperatures from the proposed equations (Eqns. 7.18-7.22) with those from FEA (Beam 200×300mm)

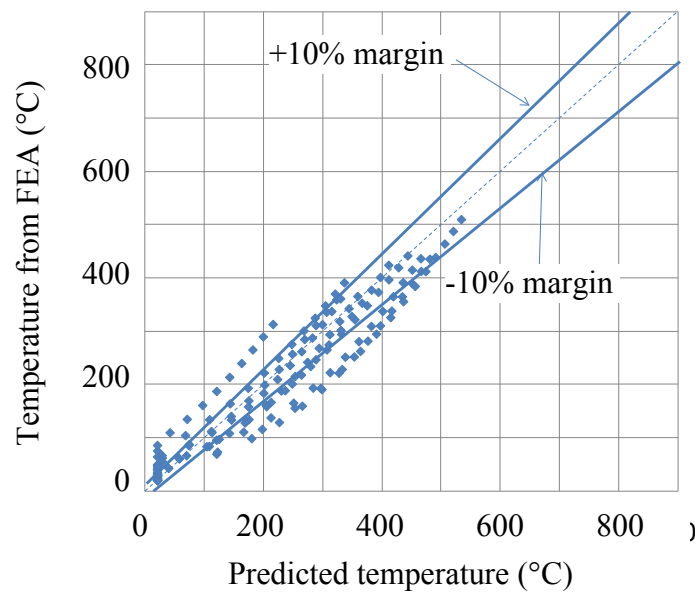


Figure 7.16 Comparison of predicted temperatures from the proposed equations (Eqns. 7.18-7.22) with those from FEA (Beam 250×400mm)

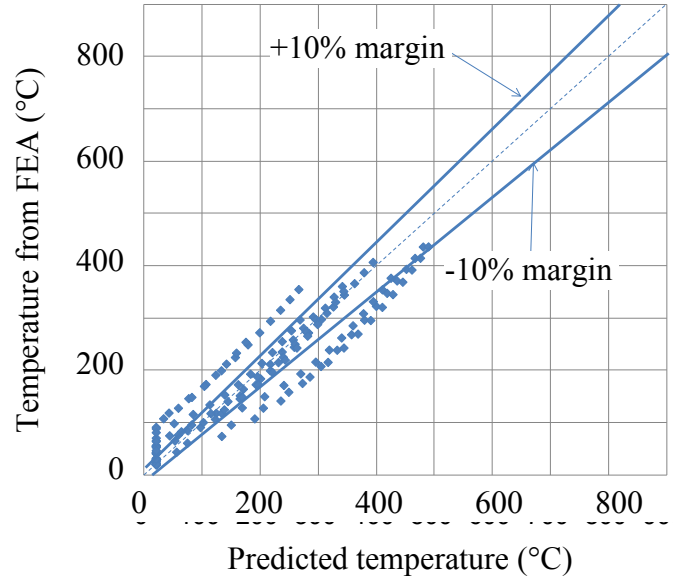


Figure 7.17 Comparison of predicted temperatures from the proposed equations (Eqns. 7.18-7.22) with those from FEA (Beam 300×500mm)

7.2.2.3 Verification of temperature equations using test results

The validity of the proposed approach is established by comparing the predicted temperatures using Eqns. 7.18-7.22 with the measured temperatures from fire tests reported in the literature. The validation focuses on the accuracy of temperature prediction in FRP and steel reinforcement, since these temperatures apply critical influence on fire response of FRP strengthened RC beams. Details of tested members selected for validation are tabulated in Table 7.4.

Figures 7.18-7.21 show the comparison of predicted steel and FRP temperatures from the proposed approach with those recorded in the fire tests. It can be seen that the predicted temperatures are mostly in good agreement with the measured values in insulated RC beams, especially on temperature predictions in steel rebar. This is very important to FRP strengthened RC members, since steel rebars provide primary

contribution to moment capacity under fire conditions. For temperatures in FRP, the predicted FRP temperatures are slightly lower than the measured ones, especially in the initial fire exposure. This is mainly due to quick rise in fire temperatures in early stage, leading to significant temperature increase in FRP at beam soffit. Also, if fire insulation cracks during the fire test (such as the case in MSU beam II), the measured FRP temperatures can suddenly jump to a high level. Since the proposed temperature equations do not account for these uncertain factors such as insulation cracking, predicted temperatures using equations are relatively lower than measured temperatures in the fire tests.

A further examination on Figures 7.18-7.21 indicates there is relatively larger discrepancy between predicted and measured temperatures in 20-100°C range. This is because this temperatures range is not primary objective in the regression analysis. As mentioned in Section 7.2.1, the regression analysis cannot match all the data closely. Thus the regression analysis was performed to fit the data points in the critical temperature range. Since temperature variation in 20-100°C range does not significantly influence the strength in steel and FRP, the accuracy of temperature predictions in this range is set as a secondary target in the regression analysis. However, this does not significantly affect further structural analysis of FRP strengthened RC beams. Overall predicted temperatures have a good agreement with measured data in most fire duration, and this demonstrates the validity of the equivalent concrete approach for predicting temperatures in an insulated RC beam.

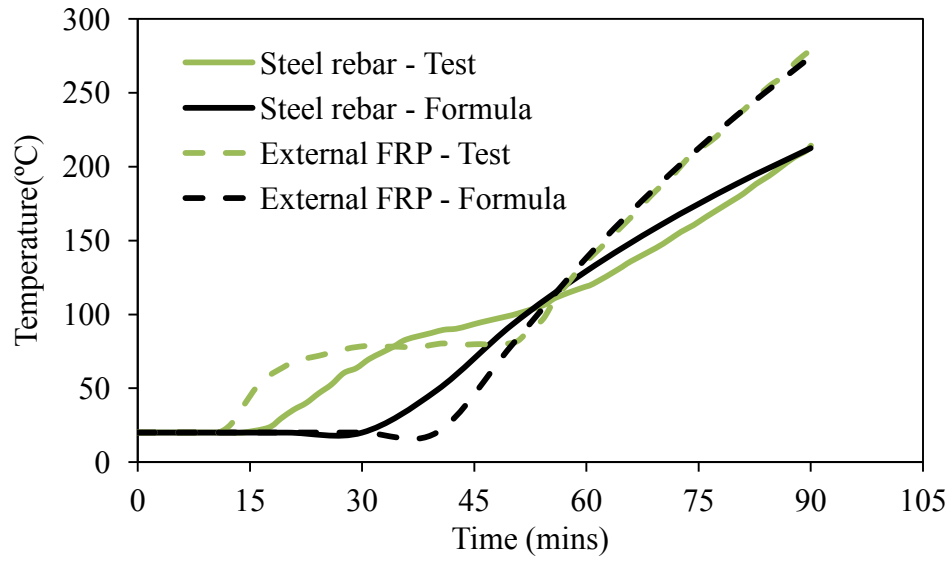


Figure 7.18 Validation of the proposed approach by comparing predicted and measured temperatures (Blontrock et al. 2000)

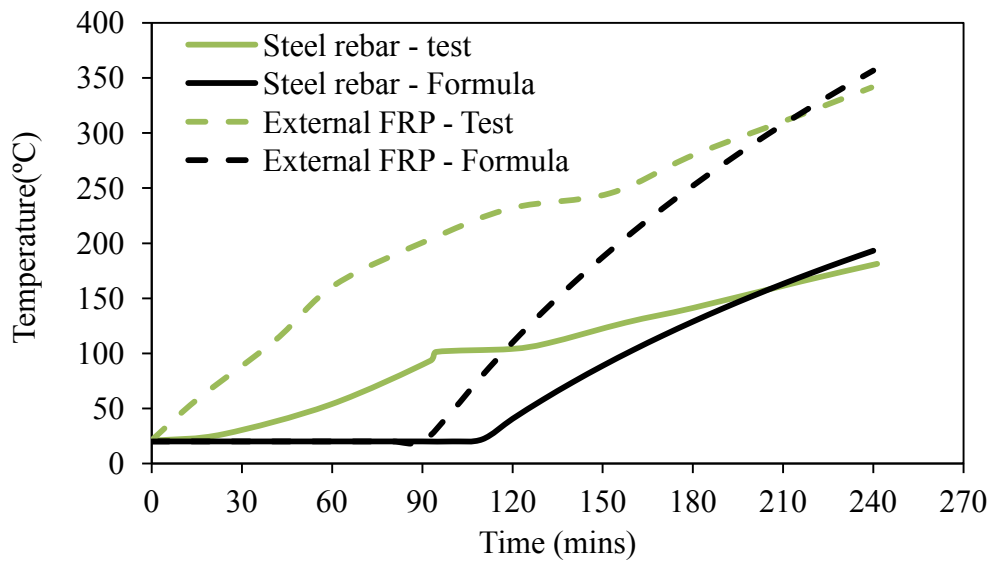


Figure 7.19 Validation of the proposed approach by comparing predicted and measured temperatures (Williams et al. 2008)

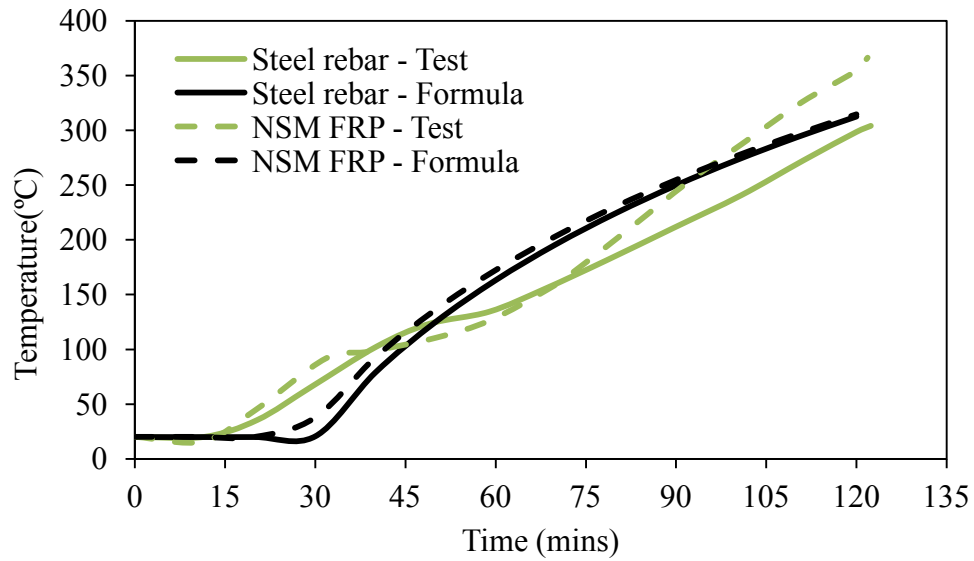


Figure 7.20 Validation of the proposed approach by comparing predicted and measured temperatures (Palmieri et al. 2012)

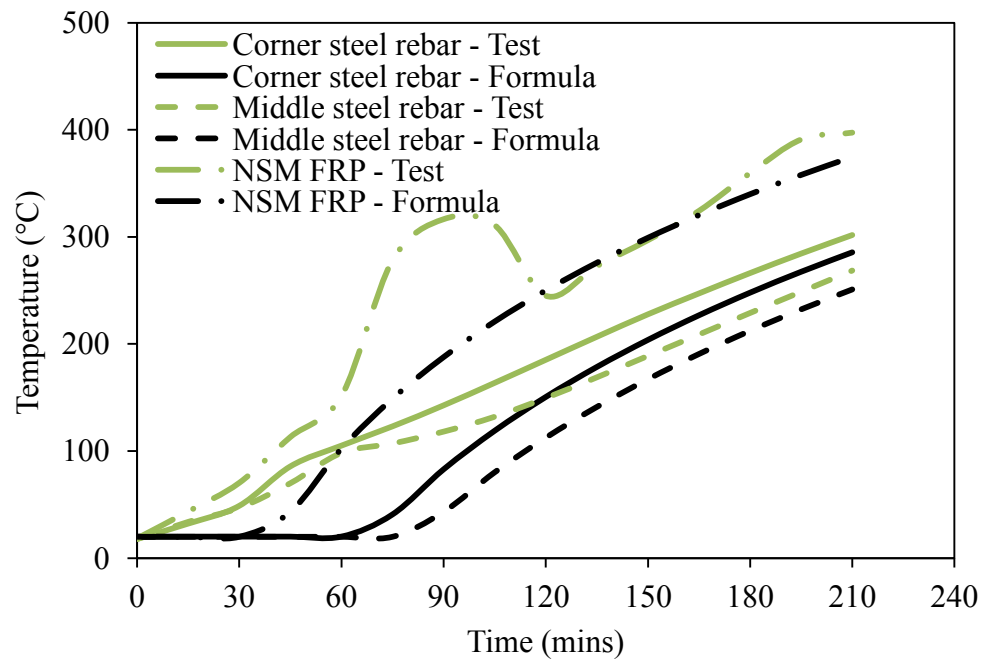
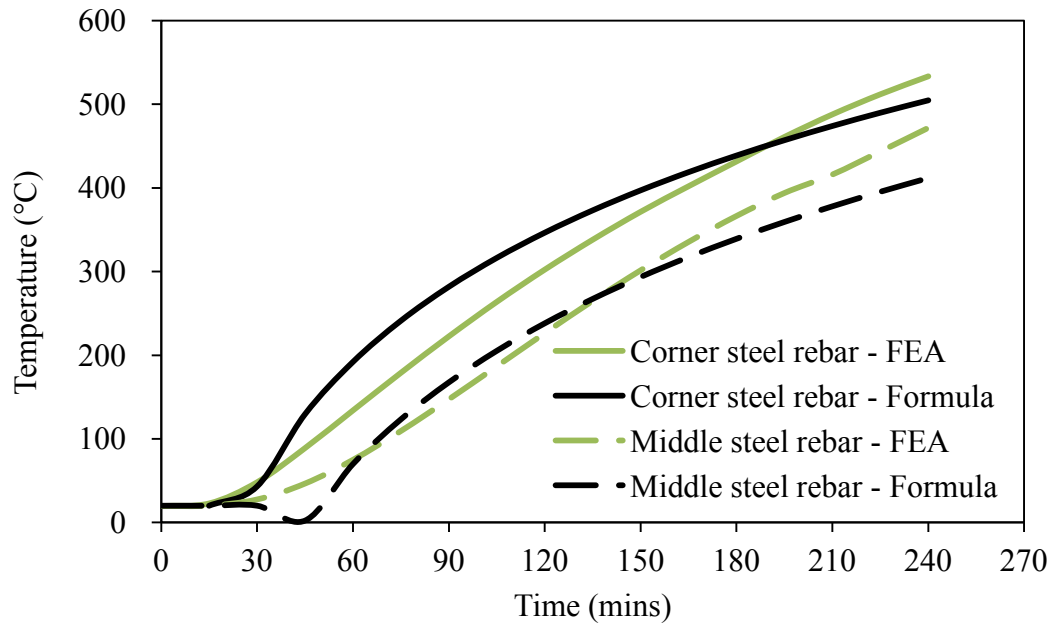


Figure 7.21 Validation of the proposed approach by comparing predicted and measured temperatures (MSU Beam II)

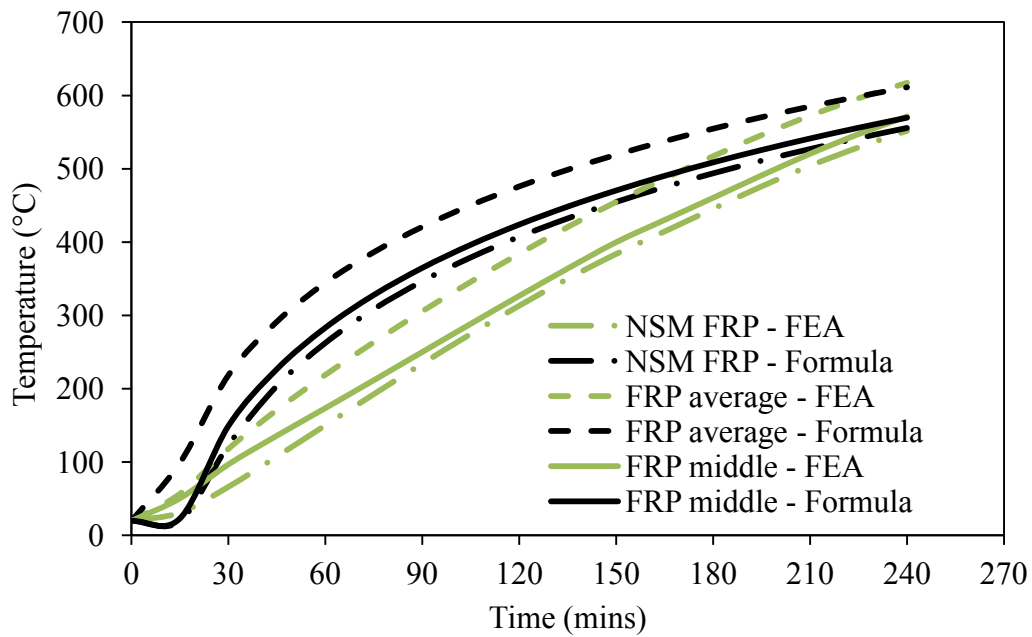
7.2.2.4 Verification of temperature equations using FEA results

The proposed equations are further validated through comparing temperature predictions with those obtained from FEA. To demonstrate the applicability of the equations in a wide range of situations, the selected concrete beams for validation are different from those used in regression analysis, as shown in Table 7.3 and Figure 7.14. The cross-sections of beams used for validation are 200×350 mm and 350×500 mm, and two types of fire insulation are applied on selected beams respectively. In each selected beam, temperatures in corner and middle steel rebars, NSM FRP strip, and center and average temperature of external FRP laminates are evaluated for comparison.

A comparison of predicted temperatures from proposed equations (Eqns. 7.18-7.22) with those from FEA is plotted in Figures 7.22 - 7.25. It can be seen that predicted temperatures reasonably match with those obtained from FEA. In a few cases, mainly in the cases of FRP laminates or strips, the predicted temperatures are conservative (higher) as compared to FEA results. This can be attributed to the fact that proposed equations do not account for the variation of thermal properties of insulation at high temperatures. Also, temperature prediction in 20-200°C range has relatively larger discrepancy with FEA results, as shown in Figure 7.25(a). This is mainly due to the fact that 20-200°C range is not primary target in the regression analysis as explained earlier. However, the discrepancy between predicted temperatures and FEA results is mostly within in 10%. Thus these equations are applicable in design situation. Overall the comparison of predicted temperatures with data from FEA results indicates the proposed equations are capable of evaluating temperatures in insulated RC beams.

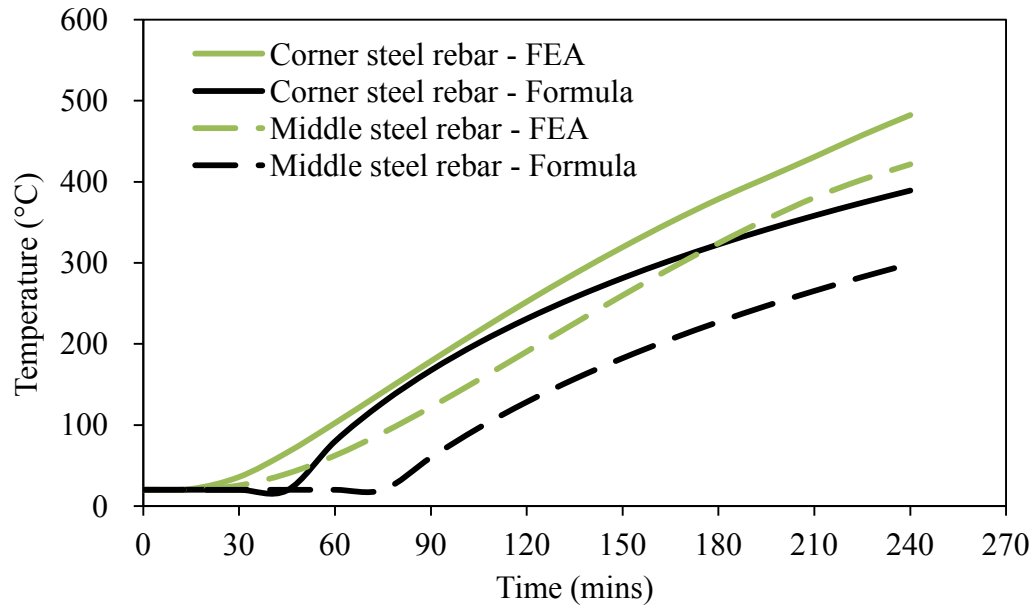


(a) Steel rebar temperatures

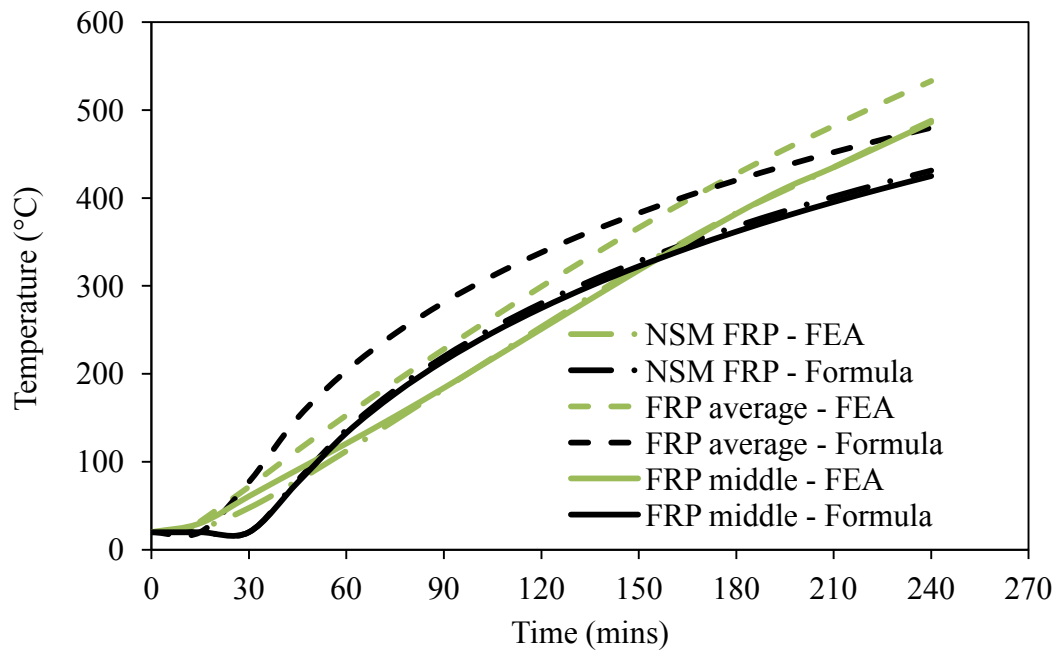


(b) FRP temperatures

Figure 7.22 Validation of the proposed approach by comparing predicted temperatures with FEA results (Beam 200×350mm with Aestver insulation)

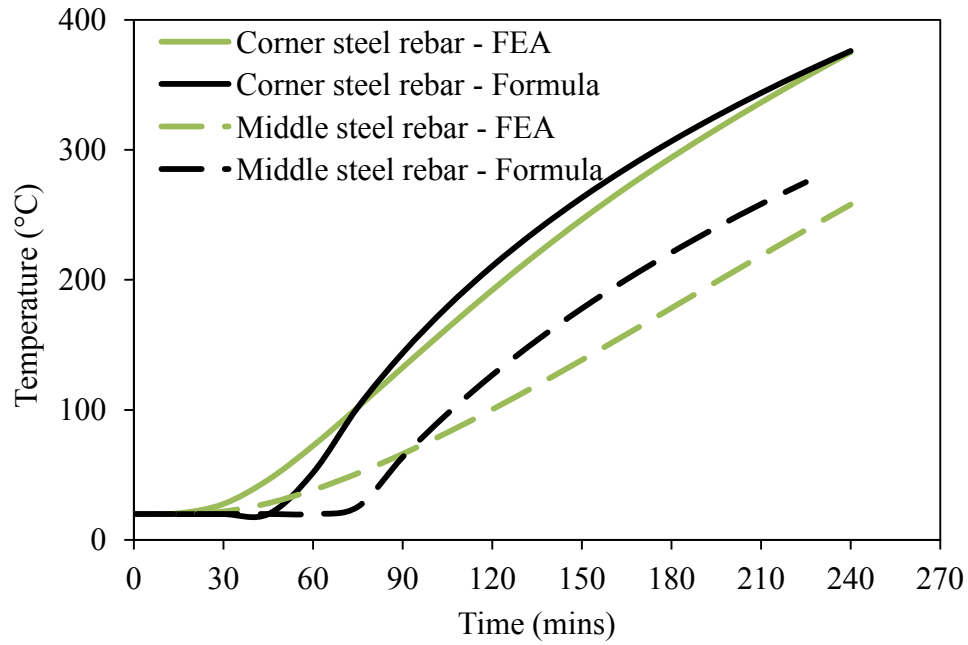


(a) Steel rebar temperatures

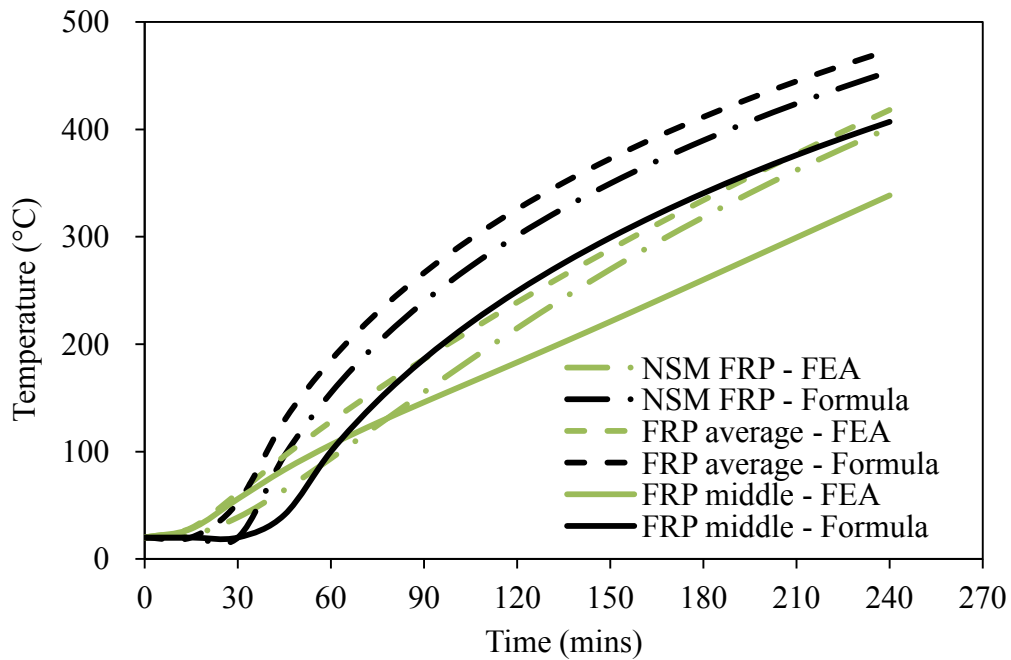


(b) FRP temperatures

Figure 7.23 Validation of the proposed approach by comparing predicted temperatures with FEA results (Beam 200×350mm with VG insulation)

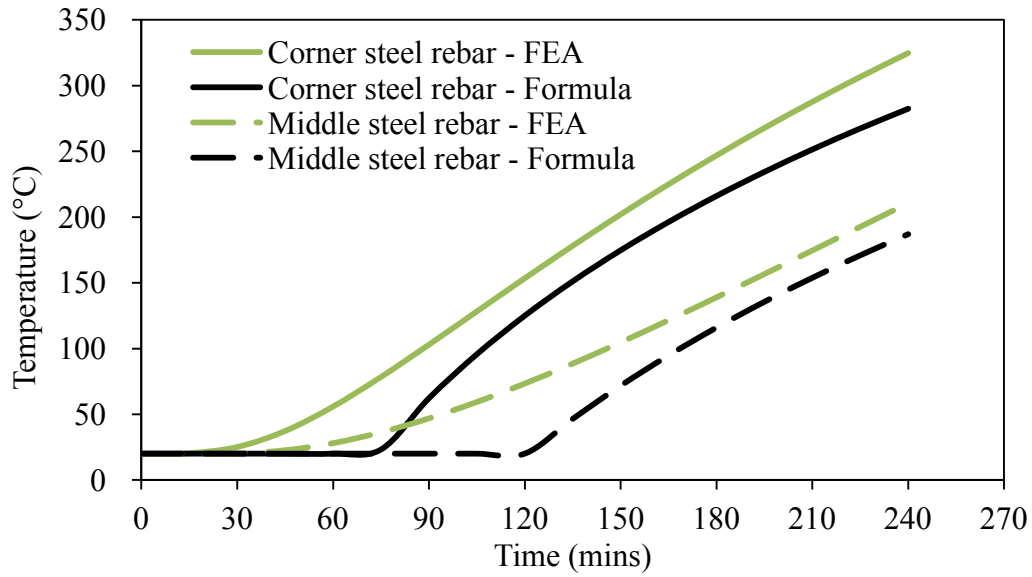


(a) Steel rebar temperatures

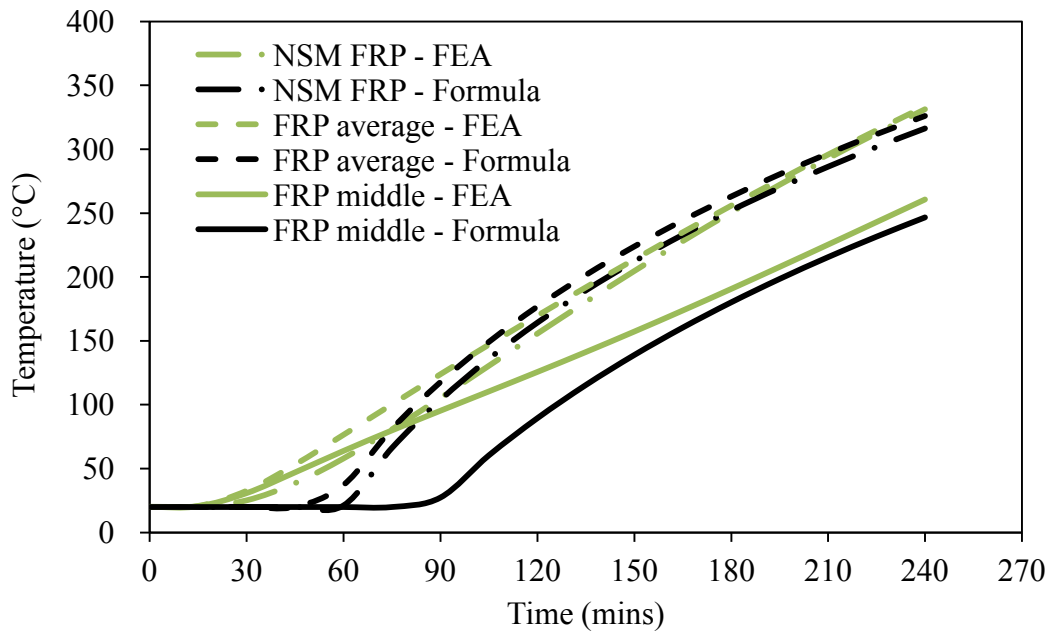


(b) FRP temperatures

Figure 7.24 Validation of the proposed approach by comparing predicted temperatures with FEA results (Beam 350×500mm with Aestver insulation)



(a) Steel rebar temperatures



(b) FRP temperatures

Figure 7.25 Validation of the proposed approach by comparing predicted temperatures with FEA results (Beam 350×500mm with VG insulation)

7.3 Evaluating Moment Capacity of FRP-strengthened RC Beams

Once temperatures in steel and FRP reinforcement are obtained, flexural capacity of fire exposed FRP strengthened RC beam can be evaluated at any given fire exposure time, utilizing similar procedure as that for room temperature moment capacity as specified in ACI 440.2 (2008). To apply these equations for evaluating moment capacity at a given fire exposure time, corresponding strength loss in concrete, FRP and steel reinforcement due to temperature rise needs to be accounted. This section provides detailed procedure for evaluating moment capacity of FRP strengthened RC beam at any give fire exposure time, and a flow chart is also provided for better illustrating step by step calculations.

7.3.1 Degradation of steel and FRP properties

Temperature induced strength degradation in steel and FRP reinforcement have significant influence on resisting moment capacity of an FRP strengthened RC beam at a given fire exposure time. For steel rebars, the degradation of strength and elastic modulus at evaluated temperature has been well studied and documented. In this research, temperature-property relations specified as per Eurocode 2 (2004), are applied for evaluating the strength and elastic modulus of steel rebar at any give temperature (or fire exposure time).

For FRP reinforcement (EBR laminates or NSM rods and strips), there is very limited information on the degradation of mechanical properties at elevated temperature. It has been known that FRP exhibits a linear stress-strain response both at room temperature (ACI 440.1 2006, FIB Bulletin 14 2007) and at elevated temperatures (Wang

et. al 2007, Bisby et al. 2005, Yu and Kodur 2013). Thus, temperature dependent stress-strain response of FRP can be represented through a set of linear relationships. In this section, it is assumed that high temperature strength and modulus of external FRP laminates follow the empirical relations proposed by Bisby et al. (2005), and those of NSM FRP follow the empirical relations proposed in Chapter 3.

The temperature dependant property relations of reinforcing steel and FRP, are summarized in Appendix A.

7.3.2 Effective concrete width under fire exposure

For an RC beam exposed to fire exposure, concrete in compression zone is not usually protected by fire insulation. Thus concrete in compression zone experiences certain level of strength and stiffness degradation, especially at the area close to fire exposed surface (sides of the beam). However, concrete in compression zone experiences different strength degradation depending on the specific location. Thus it is difficult to account for concrete strength degradation in the calculation moment capacity. To evaluate the effect of concrete degradation on moment capacity of RC beam, a method of “effective concrete width”, similar to that in Eurocode 2 (2004), is utilized in this study. This method assumes that concrete elements still possess full room temperature strength, but the width of concrete section is reduced due to high temperatures on fire exposure surface.

Although providing a concept of “effective concrete width”, Eurocode 2 (2004) does not provide specific values of effective concrete width for various concrete members. In this study, the effective concrete width is quantified over a wide range of beam

sections. For this purpose a set of RC beams (as shown in Table 7.4) were analyzed utilizing finite element program presented in Chapter 5. For each analyzed beam, the strength degradation in each concrete element in compression zone (the upper half beam) is evaluated at various fire exposure times. The average strength degradation of all these elements (in percentage) is calculated as the reduction factor of concrete width. Summary on these reduction factors for various beam sections is tabulated in Table 7.4. The effective concrete width of other beam sections can be obtained through linear interpolation on these known values.

Table 7.4 Factors for calculating effective concrete width for various RC beams
exposed to ASTM E119 standard fire

Time (minutes)	Effective width factors (%)				
	150×200	200×300	250×400	300×500	400×600
0	100	100	100	100	100
15	99.7	99.9	99.9	99.9	99.9
30	91.1	97.2	98.2	97.4	98.2
45	83.0	95.6	96.0	94.0	95.6
60	75.5	91.9	92.8	91.2	93.3
75	69.1	88.8	90.7	88.1	91.4
90	61.7	86.4	88.0	86.1	88.9
105	63.3	83.5	86.2	83.5	87.7
120	68.2	80.6	83.8	82.2	86.3
135	69.5	78.5	82.3	80.2	84.7
150	66.6	75.2	80.1	78.8	83.5
165	61.7	72.1	78.0	77.2	82.4
180	56.4	69.8	76.8	75.7	81.2
195	52.4	66.7	74.6	74.6	80.4
210	49.8	63.0	72.8	73.3	79.3
225	48.3	59.7	70.6	71.9	78.3
240	47.1	56.8	68.9	70.6	77.8

7.3.3 Evaluating moment capacity at a given fire exposure time

Knowing temperature dependent strength properties of steel rebar ($f_{y,T}$) and FRP ($f_{f,T}$, $E_{f,T}$) and effective concrete width (b_T), the moment capacity of FRP strengthened RC beams at any give fire exposure time can be calculated. Based on ACI 440.2 provisions (2008), there are two primary failure modes that occur in FRP strengthened RC beams:

- Crushing of top concrete before FRP debonding
- Rupture of FRP laminate after steel yielding

Thus moment capacity of FRP strengthened beams can be evaluated using different formulas for these two failure modes. Figure 7.26 illustrates stress and internal force on cross section of an NSM FRP strengthened RC beam.

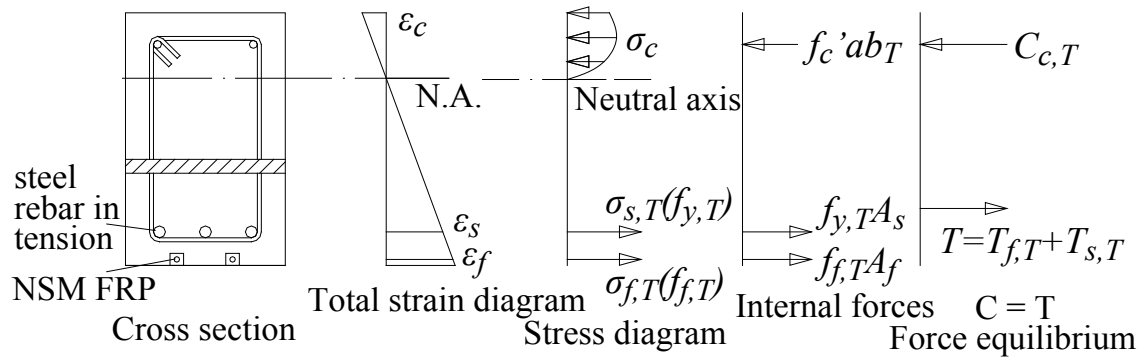


Figure 7.26 Force equilibrium and strain compatibility of NSM FRP strengthened RC beam at a given fire exposure time

At a given fire exposure time, if the beam fails due to crushing of top concrete, top concrete is considered to reach its compression strain limit ($\epsilon_c = \epsilon_{cu} = 0.003$), while strains in steel and FRP reinforcement can be calculated using the strain compatibility principles as follows

$$\frac{\varepsilon_s}{d-c} = \frac{\varepsilon_{cu}}{c} \quad (7.24)$$

$$\frac{\varepsilon_f}{h-c} = \frac{\varepsilon_{cu}}{c} \quad (7.25)$$

Since the strain hardening effect is usually neglected in the design codes, the steel stress is assumed to be that of yield strength as long as $\varepsilon_s \geq \varepsilon_{sy}$. The stress in FRP reinforcement can be calculated as

$$f_{f,T} = E_{f,T} \varepsilon_f \quad (7.26)$$

Based on the force equilibrium principle, the following equation can be obtained,

$$0.85 f_c' a b_T = A_s f_{y,T} + A_f f_{f,T} \quad (7.27)$$

ε_f and a are the only unknowns in Eqns. 7.24 - 7.27, and thus the values of ε_f and a can be identified.

Then the flexural moment capacity at a given fire exposure time can be calculated as

$$M_{n,T} = A_s f_{y,T} (d - 0.5a) + A_f f_{f,T} (h - 0.5a) \quad (7.28)$$

If the beam fails due to rupture of FRP reinforcement, FRP reaches its strain limit ($\varepsilon_f = \varepsilon_{fe} = \kappa_m \varepsilon_{fu}$). κ_m is strain reduction factor specified in ACI 440.2 (2008) to prevent the debonding failure of FRP reinforcement. Equating tension and compression forces,

$$0.85 f_c' a b_T = A_s f_{y,T} + A_f (E_{f,T} \varepsilon_{fe}) \quad (7.29)$$

Solving Eq. 7.29 for the depth of the equivalent rectangular stress block a ,

$$a = \frac{A_s f_{y,T} + A_f (E_{f,T} \varepsilon_{fe})}{0.85 f_c' b_T} \quad (7.30)$$

Then the flexural moment capacity of FRP strengthened beam can be obtained using

$$M_{n,T} = A_s f_{y,T} (d - 0.5a) + A_f (E_{f,T} \varepsilon_{fe}) (h - 0.5a) \quad (7.31)$$

For an FRP strengthened RC beam exposed to fire, the remaining strength of constituent materials is known for any given fire exposure time, based on the above described procedures. However, a preliminary calculation is needed to decide whether the moment capacity is governed by crushing of top concrete or rupture of FRP. In this preliminary calculation, Eq. 7.27 is applied firstly to obtain the equivalent stress block a . Then the strain level in FRP can be evaluated using

$$\varepsilon_f = \frac{\varepsilon_{cu}}{c} (h - c) \quad (7.32)$$

If $\varepsilon_f > \varepsilon_{fe}$, FRP reinforcement will reach its strain limit before crushing of top concrete, and thus the rupture of FRP governs the failure mode, and the moment capacity of the beam can be calculated using Eqns. 7.29-7.31. While if $\varepsilon_f < \varepsilon_{fe}$, crushing of top concrete governs the failure mode, and the moment capacity of the beam can be calculated using Eqns. 7.24-7.28.

Knowing moment capacity of FRP strengthened beams at any given fire exposure time, the fire resistance of the beam can be evaluated by comparing moment capacity with applied moment resulting from external loading. At room temperature, considering only dead loads and live loads for the strength limit state, most design codes (ASCE 7 2010, Eurocode 1 2002) specify two load combinations as follows

$$M_u = 1.4D \quad (7.33)$$

or
$$M_u = 1.2D + 1.6L \quad (7.34)$$

where, M_u is the ultimate load (moment) resulting from factored dead and live loads. D is the dead load, L is the live load.

However, in the event of fire, the applied loading is much lower than the maximum design loads specified for ambient conditions, since fire is a rare (accidental) event. ASCE 7 (2010) recommends loading under fire conditions to be evaluated as

$$M_{fire} = 1.2D + 0.5L \quad (7.35)$$

Where, M_{fire} is the loading (moment) under fire exposure.

Thus combining Eqns. 7.24-7.35, the following criteria should be applied to evaluate failure of FRP strengthened RC beam:

$$\text{At room temperature: } M_u \leq \phi M_n \quad (7.36)$$

$$\text{Under fire exposure: } M_{fire} \leq M_{n,T} \quad (7.37)$$

where, ϕ is the strength reduction factor specified in ACI 440.2R (2008). Under fire conditions, no reduction factor is applied.

Utilizing the above simplified equations, moment capacity of an FRP strengthened RC beam can be calculated at any given fire exposure time. When the moment due to external loading exceeds moment capacity, the beam fails under fire conditions, and the corresponding time is the fire resistance of an FRP strengthened beam. A flowchart illustrating the approach for evaluating fire resistance of an FRP strengthened beam is plotted in Figure 7.27. To further illustrate detailed procedure of the proposed approach, two design examples on fire resistance evaluation, one NSM FRP strengthened RC beam without insulation and one external FRP strengthened RC beam with insulation, are provided in Appendix D.

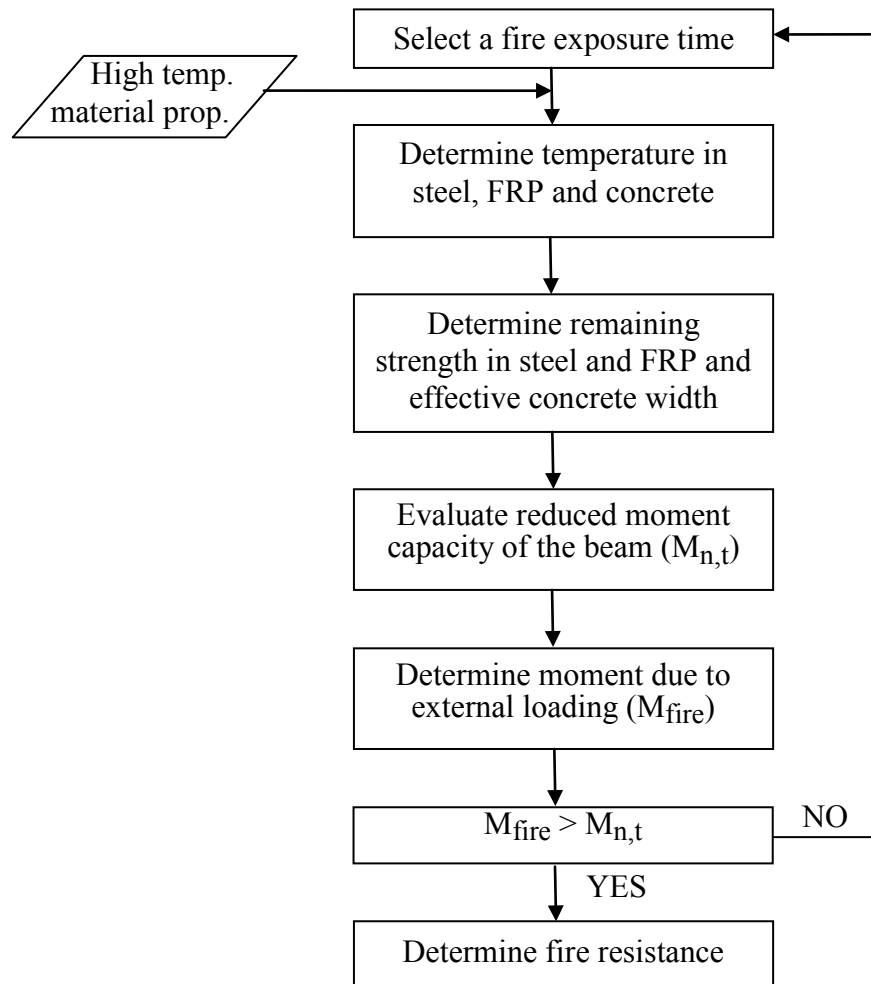


Figure 7.27 A flowchart illustrating rational design approach for evaluating fire resistance of FRP strengthened beam

7.4 Validation of the Proposed Approach

The validity of the above proposed approach is established by comparing the predicted response of beams with results from fire tests and FEA. To demonstrate usefulness of the proposed approach, the selected FRP strengthened beams cover those with various strengthening types (NSM or EBR) and fire protection (with and without insulation). Details on these selected beams are tabulated in Table 7.5.

The comparison of predicted fire response with those from fire tests is plotted in Table 7.5. Since the variation of moment capacity with fire exposure time cannot be

directly measured in fire tests, the measured fire resistance (failure times) of tested beams is compared with those predicted from the proposed approach. It can be seen in Table 7.5 that the proposed approach provides good predictions on fire resistance of insulated RC beams with FRP strengthening, while predictions in the uninsulated beams are relatively conservative. In the uninsulated beams selected for comparison (Firmo et al. 2012, MSU Beam I in Chapter 4), the anchorage area of FRP laminates or strips was protected by furnace walls. Thus during the fire tests, a cable action was developed through cool ends of FRP, and this provided extra support to RC beams even FRP reached very high temperatures. The proposed approach does not account for this cable action effect, and thus predicts a lower fire resistance. In fact, since the extra moment capacity or fire resistance resulting from the cable action depends on a number of factors, and it is almost impossible to quantify its influence. Thus, the fire resistance predictions from the proposed approach are appropriate to be used in design situations.

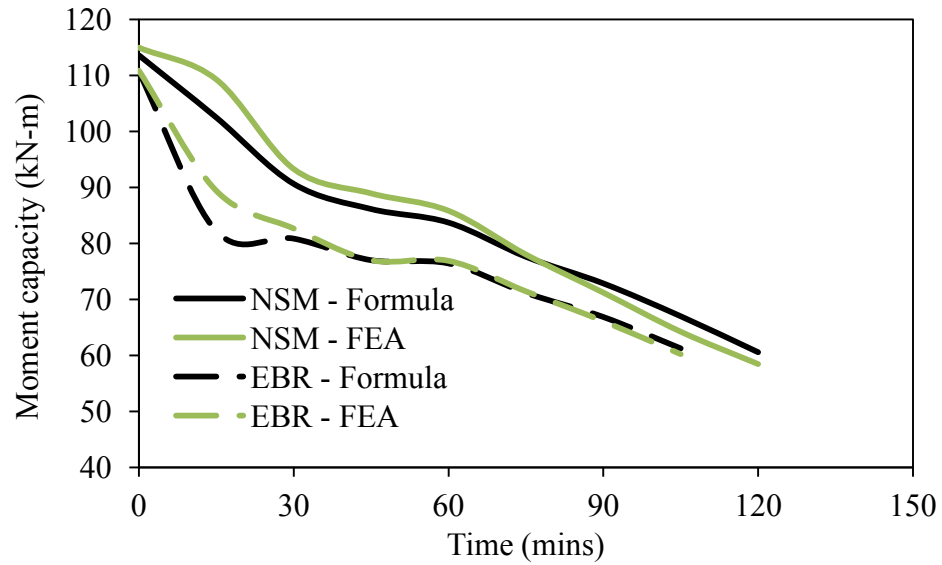
The predictions from proposed approach are further validated through comparing the predicted fire response with those obtained from FEA results. The beams in section of 200×350mm and 350×500mm were analyzed in finite element program to compare their moment capacities with predictions using proposed approach, as shown in Figures 7.27-7.28. It can be seen that the predicted moment capacity is usually slightly lower than those from FEA results. This can be attributed to rough estimation on effective concrete width as well as conservative predictions on steel and FRP temperatures. Thus the discrepancy between predicted and analyzed moment capacity is reasonable. Moreover, the predicted fire resistance well matches FEA results, as shown in Table 7.5.

The calculation using proposed approach indicates that at ambient conditions, crushing on top concrete usually governs the failure of beams, for both EBR and NSM strengthening. This is because FRP laminates or strips possess quite large tensile strength and modulus, and top concrete reaches its failure strain (0.003) before FRP breaks. While at high temperature, FRP reinforcement usually loses most of strength, and strengthened beams usually develop relatively larger curvatures. Thus, FRP reinforcement easily reaches its failure strain before top concrete crushing, and rupture of FRP governs the failure of strengthened beams.

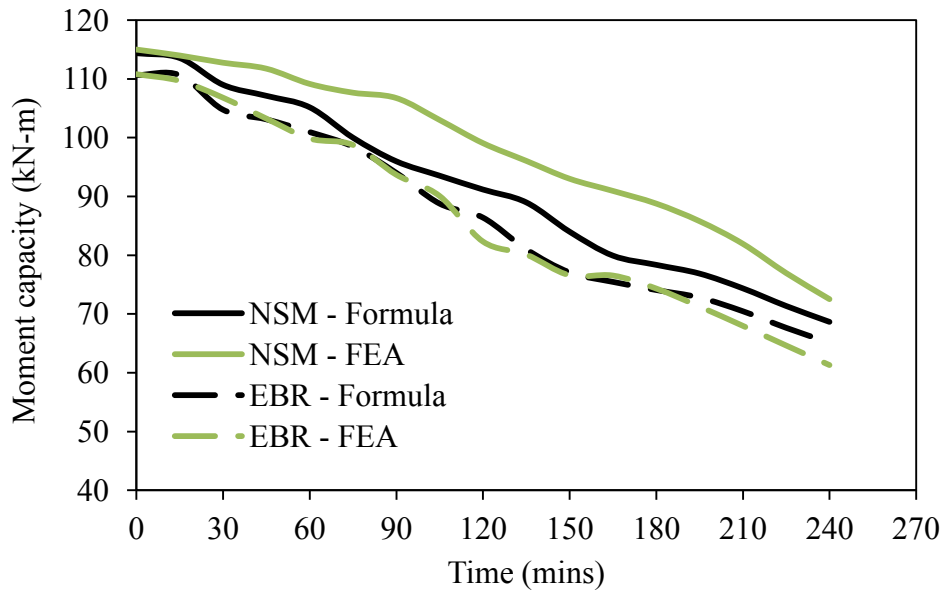
Table 7.5 Comparison of fire resistance using proposed approach with
fire tests and FEA results

Source of data	Selected beam	FRP strengthening	Fire insulation	Fire resistance (mins)	
				Test / FEA	Proposed approach
Fire tests	Firmo et al. (2012)	EBR	No	60	15
	MSU Beam I	NSM	No	>210*	180
	Blontrock et al. (2000)	EBR	Yes	>90*	>90
	Williams et al. (2008)	EBR	Yes	>240*	>240
	Palmieri et al. (2012)	NSM	Yes	>120*	>120
	MSU Beam II	NSM	Yes	>210*	>210
FEA	Beam 200×350	EBR	No	105	105
	Beam 200×350	NSM	No	125	120
	Beam 350×500	EBR	No	90	90
	Beam 350×500	NSM	No	120	120
	Beam 200×350	EBR	Yes	240	240
	Beam 200×350	NSM	Yes	240	240
	Beam 350×500	EBR	Yes	240	240
	Beam 350×500	NSM	Yes	240	240

* The beam did not fail in tests till reported time.

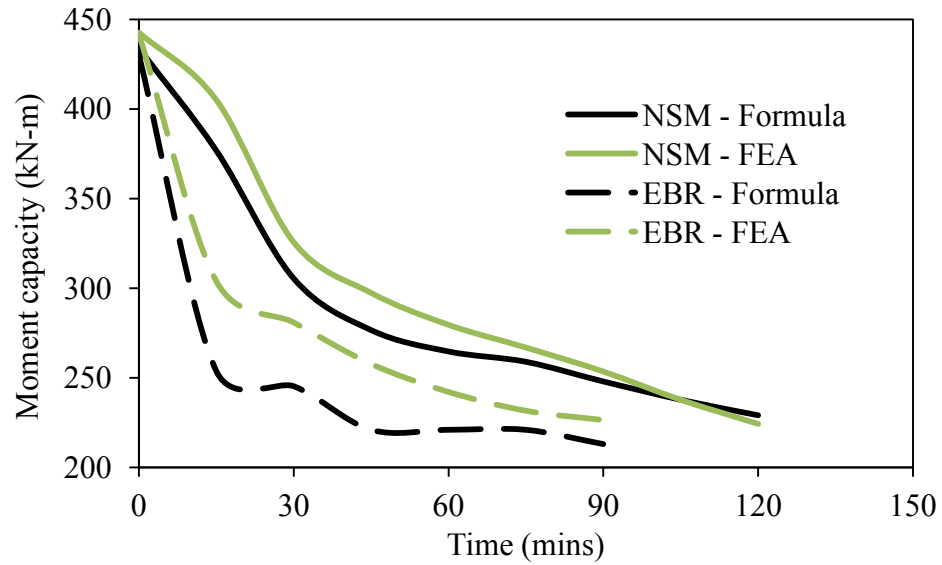


(a) Beams without insulation

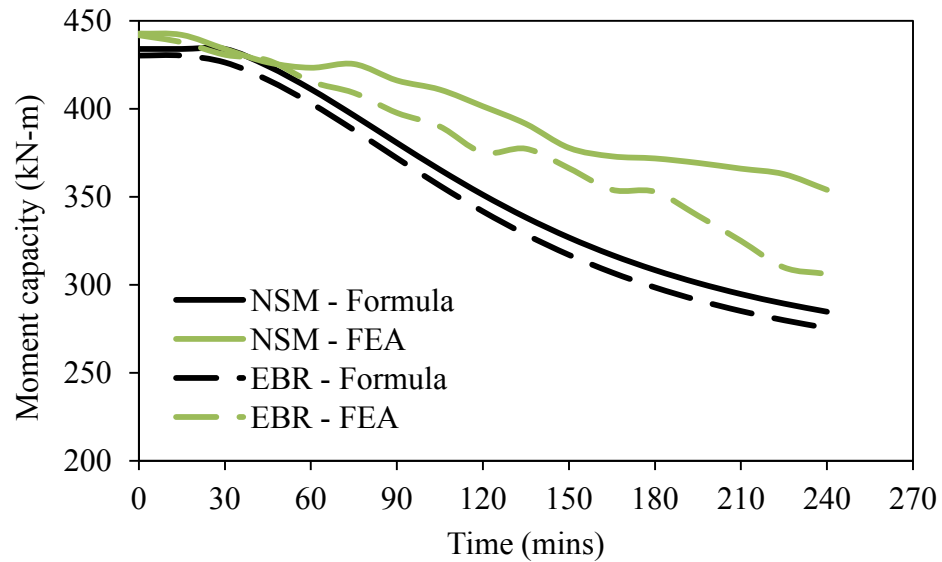


(b) Beams with insulation

Figure 7.28 Validation of the proposed approach by comparing predicted moment capacity with FEA results (Beam 200×350mm with VG insulation)



(a) Beams without insulation



(b) Beams with insulation

Figure 7.29 Validation of the proposed approach by comparing predicted moment capacity with FEA results (Beam 350×500 mm with VG insulation)

7.5 Limitations of Applicability

Although the proposed approach is applicable over a large range of parameters, the following limitations are to be applied since these equations are developed based on these types of FRP strengthened RC members in the analysis.

- 1) The proposed approach is applicable for evaluating temperatures or fire resistance in FRP strengthened RC members exposed to standard fire only. These equations are not applicable for design fires, which have a cooling phase following the growth phase.
- 2) The temperature predictions on FRP laminates or strips using proposed equations does not account for uncertainty factors influencing fire insulation, such as cracking of insulation, or uneven insulation thickness.
- 3) The proposed approach does not account for the cable action developed through cool anchorage of FRP reinforcement. In the cases where anchorage zones are protected, the proposed approach gives conservative predictions.
- 4) The proposed simplified approach for moment capacity is applicable to simply supported RC beams only, since the effect of axial restraint resulting from fire is not taken into account.

7.6 Summary

This chapter presents a simplified approach for assessing fire resistance of FRP strengthened RC beams exposed to standard fire. This approach is developed by applying an analogy as that of room temperature design as specified in ACI 440.2 (2008), but the temperature induced strength degradation in concrete, reinforcing steel, and FRP, are accounted for in evaluating moment capacity at any give fire exposure time. The

proposed approach is capable of predicting temperatures at various locations, and it also accounts for various parameters such as FRP strengthening type and insulation properties. The validity of the proposed approach is established by comparing temperature and fire resistance predictions with those obtained from fire tests and finite element analysis. The applicability of the proposed approach in design situation is also illustrated through detailed examples. Overall the proposed approach provides a simple and rational method for evaluating fire response of FRP strengthened RC beams exposed to standard fires.

CONCLUSIONS AND RECOMMENDATIONS

8.1 General

This dissertation presents a comprehensive study on the behaviour of NSM FRP strengthened RC beams under fire conditions. Both experimental and numerical studies were carried out to evaluate the fire resistance of NSM FRP strengthened RC beams and the critical influencing factors. As part of experimental studies, a large number of property tests were carried out to develop data on variation of thermal and mechanical properties of NSM FRP as a function of temperature. Data from these tests was utilized to develop empirical temperature-property relations for NSM FRP over a wide temperature range. Further, full-scale fire resistance tests were carried out on four NSM FRP strengthened RC T-beams. Data from fire tests was utilized to gauge the effect of insulation, load level, and axial restraint on fire resistance of NSM FRP strengthened beams.

As part of numerical studies, a numerical model, previously developed for externally bonded FRP strengthened RC beams, was extended to model the response of NSM FRP strengthened RC beams under realistic fire, loading and restraint conditions. This model is based on a macroscopic finite element approach, and utilizes time dependent moment-curvature relationships to trace the response of FRP strengthened RC beam from pre-loading to failure under fire conditions. The model accounts for high temperature properties of constituent materials, various strain components, and fire

induced bond degradation and axial restraint force. The validity of the model was established by comparing predicted response parameters with measured data in fire tests carried out at MSU and also those reported in literature.

The validated numerical model was further applied to conduct a set of parametric studies to quantify the influence of critical factors on fire response of NSM FRP strengthened RC beams. Results generated from parametric studies were utilized to develop a rational design methodology for evaluating fire resistance of NSM FRP strengthened RC beams. This methodology comprised of two steps, namely, evaluating cross-sectional temperatures and calculating moment capacity of beam at any given fire exposure time. For accessing temperature profiles in an FRP strengthened RC beam, a set of empirical temperature equations was developed by taking into account various influencing factors. For accessing moment capacity of FRP strengthened RC beams at any given fire exposure time, an approach similar to that at room temperature is utilized but temperature dependant strength properties of concrete, steel and FRP are substituted in place of room temperature properties. The proposed approach accounts for various factors influencing fire response of FRP strengthened RC beams, and thus provides a useful tool for structural fire design of FRP strengthened RC beams.

8.2 Key Findings

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

1. There is very limited information on fire response of NSM FRP strengthened RC beams. Since the behavior of NSM FRP strengthened RC members is quite

- different from that of external FRP strengthened RC members, currently available data for external FRP strengthened beams cannot be directly applied to NSM FRP strengthened beams.
2. NSM CFRP strips and rods retain much of tensile strength and modulus (up to 80%) till about 200°C. Beyond 200°C, tensile strength and elastic modulus of NSM CFRP decrease at a faster pace due to decomposition of polymer resin. At 600°C, NSM CFRP retains only about 10% of its original strength.
 3. Bond strength and modulus of NSM FRP system decrease significantly with temperature, and only 20% of the original bond strength is retained at 200°C. Bond strength and modulus continue to degrade in 200-400°C range, and reaches almost zero at 400°C. Bond stress-slip response of NSM FRP follows a similar pattern at both room and high temperatures. However, the peak value (bond strength) and the slope (bond modulus) are smaller at high temperatures.
 4. Coefficient of thermal expansion (CTE) of NSM FRP varies significantly depending on direction and composition. NSM GFRP has positive CTE (expansion) in both transverse and longitudinal directions. NSM CFRP expands in transverse direction at elevated temperatures, but shrinks in longitudinal direction. GFRP and CFRP experience larger thermal expansion (or shrinking) at higher temperatures.
 5. NSM FRP strengthened beam can provide about three hours of fire resistance, even without fire insulation. Presence of cooler anchorage enables the remaining NSM FRP strips (or carbon fibers) to contribute to load carrying capacity of the beam through a “cable” action under fire conditions. In addition, provision of fire

insulation or axial restraint enhances fire resistance of NSM FRP strengthened RC beams.

6. The proposed macroscopic finite element model is capable of tracing the response of NSM FRP strengthened RC beams from pre-fire stage to collapse under fire conditions. The model accounts for rectangular and T cross sections, high temperature properties of constitutive materials (concrete, steel rebars, NSM FRP and insulation), bond degradation at NSM FRP-concrete interface, as well as axial restraint at beam supports. Thermal and structural response predictions from the model compare well with those measured in tests at both ambient and fire conditions.
7. Results from parametric studies indicate that type of FRP strengthening, reinforcement ratio of steel and FRP, load level, axial restraint, fire scenario, and insulation scheme have significant influence on the fire response of NSM FRP strengthened RC beam. However, location of NSM FRP and compressive strength of concrete only have moderate influence on fire response. Specifically:
 - NSM FRP strengthened RC beam possesses higher fire resistance than that of external FRP strengthened RC beam, but lower fire resistance than that of conventional RC beam.
 - Higher reinforcement ratio of FRP leads to lower fire resistance of NSM FRP strengthened RC beam.
 - Type of fire exposure has significant influence on the fire resistance of NSM FRP strengthened RC beam. Under most design fire scenarios, NSM FRP

strengthened RC beam can sustain up to three hours of fire resistance even without fire insulations.

- Higher load level lowers the fire resistance of NSM FRP strengthened RC beam. Presence of axial restraint enhances fire resistance.
 - Provision of fire insulation significantly increases fire resistance, and an optimal fire insulation scheme is also proposed for an NSM FRP strengthened RC beam.
 - Placing NSM FRP at inner locations (center of beam soffit) or using higher concrete strength slightly increases the fire resistance of NSM FRP strengthened RC beam.
8. The proposed approach for evaluating temperature and moment capacity can be applied to assess fire response of NSM FRP strengthened RC beams under standard fire exposure. This approach is capable of predicting cross sectional temperatures, stress levels in steel and FRP, effective concrete width, and moment capacity at any given fire exposure time. The simplicity and wide range of applicability of the proposed approach make it attractive for incorporation in design standards.

8.3 Recommendations for Future Research

Although this study has advanced the state-of-the-art with respect to fire response of NSM FRP strengthened RC beams, further research is required to fully characterize the complex behaviour of NSM FRP strengthened RC beams exposed to fire. The following are some of the key recommendations for future research in this area:

- Due to a large variety of available NSM adhesive materials, more experimental data is needed to evaluate the effect of NSM adhesive, especially cementitious based adhesives on fire response of NSM FRP strengthened RC beams.
- Further fire resistance experiments are needed to develop data on fire response of NSM FRP strengthened RC beams with different configurations including cross sectional type, reinforcement ratio of NSM FRP and steel, and insulation layout.
- Further experimental and numerical studies are needed to obtain complete understanding on the cable action developed through NSM FRP. The moment contribution from this cable action needs to be quantified for evaluating fire resistance of NSM FRP strengthened RC beams.
- Further work is required to incorporate more advanced features into numerical model, such as accounting for cracking in insulation or concrete, effect of FRP charring resulting from pyrolysis process, as well as accounting for prestressed concrete members with FRP reinforcement.
- More work is needed to extend the proposed design methodology to account for the response of FRP strengthened RC beams exposed to design fire scenario (with cooling phase).

8.4 Research Impact

Due to sensitivity of FRP materials to high temperatures, the performance of FRP strengthened RC members under fire conditions is a primary concern for applications in buildings, where fire resistance of a structural member is a major design requirement. Recently emerged NSM FRP strengthening is considered to possess higher fire resistance

than that of conventional strengthening methods such as externally bonded FRP. However, there is very limited information on fire response of NSM FRP strengthened members. The studies presented in this dissertation provide a comprehensive evaluation on the behavior of NSM FRP strengthening under fire conditions, from material level to structural behaviour of strengthened beams. A fundamental understanding on fire response of NSM FRP strengthened RC beams is established, and the effects of critical influencing factors, such as tensile and bond strength properties, loading and restraint conditions, are quantified through experimental and numerical studies. These studies have indicated that NSM FRP strengthened RC beams have higher fire resistance than that of external FRP strengthened RC beams. With proper design and protection, NSM FRP strengthened RC beam without any fire protection can achieve fire endurance for more than three hours. For the cases with fire insulation, NSM FRP strengthened RC beam can achieve up to four hours of fire resistance.

In addition, the numerical model presented in this study provides an effective alternative for evaluating fire response of NSM FRP strengthened RC beams. This model accounts for critical influencing factors such as high temperature properties of constituent materials and bond degradation at FRP-concrete interface. Thus the model can be used to perform detailed fire resistance analysis on NSM FRP strengthened RC beams.

Further, a rational design approach, in form of a set of simplified equations, is proposed in this study. This approach is capable of predicting various response parameters such as cross sectional temperatures and moment capacity of beam at any given fire exposure time. Thus, a quick and reliable evaluation on fire resistance of FRP strengthened RC beams can be obtained even without conducting complex fire tests or

finite element analysis. The applicability of this approach covers a wide range of beam sections and insulation schemes, so it is attractive for incorporation in codes and standards. Overall the research presented in this dissertation facilitates wider use of FRP in strengthening of concrete members in buildings and other structures where fire safety is a major design requirement.

APPENDICES

APPENDIX A

Material Properties at Elevated Temperatures

This appendix provides a summary of high temperature property relationships of constituent materials of FRP strengthened beam, including concrete, reinforcing steel, FRP, epoxy, and insulation. The information presented here is used in numerical model and parametric studies.

A.1 Concrete

A.1.1 Thermal properties

A.1.1.1 Thermal capacity ($\rho_c, T C_{c,T}$)

The following property-temperature relations are proposed by Lie (1992) in ASCE handbook.

For siliceous aggregate concrete, with T_c in °C and $\rho_c, T C_{c,T}$ in J/m³-°C

$$0 \leq T_c \leq 200: \rho_c, T C_{c,T} = (0.005T_c + 1.7) \times 10^6$$

$$200 \leq T_c \leq 400: \rho_c, T C_{c,T} = 2.7 \times 10^6$$

$$400 \leq T_c \leq 500: \rho_c, T C_{c,T} = (0.013T_c - 2.5) \times 10^6$$

$$500 \leq T_c \leq 600: \rho_c, T C_{c,T} = (-0.013T_c + 10.5) \times 10^6$$

$$T_c \geq 600: \rho_c, T C_{c,T} = 2.7 \times 10^6$$

For carbonate aggregate concrete, with T_c in °C and $\rho_{c,T}C_{c,T}$ in J/m³-°C

$$0 \leq T_c \leq 400: \rho_{c,T}C_{c,T} = 2.566 \times 10^6$$

$$400 \leq T_c \leq 410: \rho_{c,T}C_{c,T} = (0.1765T_c - 68.034) \times 10^6$$

$$410 \leq T_c \leq 455: \rho_{c,T}C_{c,T} = (-0.05043T_c + 25.00671) \times 10^6$$

$$455 \leq T_c \leq 500: \rho_{c,T}C_{c,T} = 2.566 \times 10^6$$

$$500 \leq T_c \leq 635: \rho_{c,T}C_{c,T} = (0.01603T_c - 5.44881) \times 10^6$$

$$635 \leq T_c \leq 715: \rho_{c,T}C_{c,T} = (0.005T_c - 100.90225) \times 10^6$$

$$715 \leq T_c \leq 785: \rho_{c,T}C_{c,T} = (-0.22103T_c + 176.07343) \times 10^6$$

$$T_c \geq 785: \rho_{c,T}C_{c,T} = 2.566 \times 10^6$$

For lightweight concrete, with T_c in °C and $\rho_{c,T}C_{c,T}$ in J/m³-°C

$$0 \leq T_c \leq 400: \rho_{c,T}C_{c,T} = 1.930 \times 10^6$$

$$400 \leq T_c \leq 420: \rho_{c,T}C_{c,T} = (0.0772T_c - 28.95) \times 10^6$$

$$420 \leq T_c \leq 435: \rho_{c,T}C_{c,T} = (-0.1029T_c + 46.706) \times 10^6$$

$$435 \leq T_c \leq 600: \rho_{c,T}C_{c,T} = 1.930 \times 10^6$$

$$600 \leq T_c \leq 700: \rho_{c,T}C_{c,T} = (0.03474T_c - 18.9140) \times 10^6$$

$$710 \leq T_c \leq 720: \rho_{c,T}C_{c,T} = (-0.1737T_c + 126.994) \times 10^6$$

$$T_c \geq 720: \rho_{c,T} c_{c,T} = 1.930 \times 10^6$$

The following property-temperature relations are recommended in Eurocode 2 (2004).

The variations of specific heat (J/kg-°C) of concrete with temperature (T_c in °C) are

$$20 \leq T_c \leq 100: c_{c,T} = 900$$

$$100 \leq T_c \leq 200: c_{c,T} = 900 + (T_c - 100)$$

$$200 \leq T_c \leq 400: c_{c,T} = 1000 + (T_c - 200) / 2$$

$$400 \leq T_c \leq 1200: c_{c,T} = 1100$$

The variations of density (kg/m³) of concrete with temperature (T_c in °C) are

$$20 \leq T_c \leq 115: \rho_{c,T} = \rho(20^\circ\text{C})$$

$$115 \leq T_c \leq 200: \rho_{c,T} = \rho(20^\circ\text{C})(1 - 0.02(T - 115) / 85)$$

$$200 \leq T_c \leq 400: \rho_{c,T} = \rho(20^\circ\text{C})(0.98 - 0.03(T - 200) / 200)$$

$$400 \leq T_c \leq 1200: \rho_{c,T} = \rho(20^\circ\text{C})(0.95 - 0.07(T - 400) / 800)$$

A.1.1.2 Thermal conductivity ($k_{c,T}$)

The following property-temperature relations are proposed by Lie (1992) in ASCE design handbook.

For siliceous aggregate concrete, with T_c in °C and $k_{c,T}$ in W/m-°C

$$0 \leq T_c \leq 800: k_{c,T} = -0.000625T_c + 1.5$$

$$T_c \geq 800: k_{c,T} = 1.0$$

For carbonate aggregate concrete, with T_c in °C and $k_{c,T}$ in W/m-°C

$$0 \leq T_c \leq 293: k_{c,T} = 1.355$$

$$T_c \geq 293: k_{c,T} = -0.001241T_c + 1.7162$$

For lightweight concrete, with T_c in °C and $k_{c,T}$ in W/m-°C

$$0 \leq T_c \leq 600: k_{c,T} = -0.00039583T_c + 0.925$$

$$T_c \geq 600: k_{c,T} = 0.6875$$

These property-temperature relations are recommended in Eurocode 2 (2004). The variations of thermal conductivity (W/m-°C) of concrete with temperature (T_c in °C) are

$$\text{Upper limit: } 20 \leq T_c \leq 1200: k_{c,T} = 2 - 0.2451(T_c / 100) + 0.0107(T_c / 100)^2$$

$$\text{Lower limit: } 20 \leq T_c \leq 1200: k_{c,T} = 1.36 - 0.136(T_c / 100) + 0.0057(T_c / 100)^2$$

A.1.1.3 Thermal strain ($\varepsilon_{c,th}$)

The thermal strain in concrete specified in ASCE Handbook (Lie 1992) is

$$\varepsilon_{c,th} = [0.004(T_c^2 - 400) + 6(T_c - 20)] \times 10^{-6}$$

The thermal strain in concrete specified in Eurocode 2 (2004) is

Siliceous aggregate:

$$20 \leq T_c \leq 700: \varepsilon_{c,th} = -1.8 \times 10^{-4} + 9 \times 10^{-6} T_c + 2.3 \times 10^{-11} T_c^3$$

$$700 \leq T_c \leq 1200: \varepsilon_{c,th} = 14 \times 10^{-3}$$

Carbonate aggregate:

$$20 \leq T_c \leq 805: \varepsilon_{c,th} = -1.2 \times 10^{-4} + 6 \times 10^{-6} T_c + 1.4 \times 10^{-11} T_c^3$$

$$805 \leq T_c \leq 1200: \varepsilon_{c,th} = 12 \times 10^{-3}$$

A.1.2 Mechanical properties

A.1.2.1 Stress-strain relationship

The stress-strain relationship of concrete specified in ASCE handbook (Lie 1992) is

$$\sigma_c = \begin{cases} f'_{c,T} \left[1 - \left(\frac{\varepsilon - \varepsilon_{\max,T}}{\varepsilon_{\max,T}} \right)^2 \right], & \varepsilon \leq \varepsilon_{\max,T} \\ f'_{c,T} \left[1 - \left(\frac{\varepsilon_{\max,T} - \varepsilon}{3\varepsilon_{\max,T}} \right)^2 \right], & \varepsilon > \varepsilon_{\max,T} \end{cases}$$

$$f'_{c,T} = \begin{cases} f'_c & 20^\circ\text{C} \leq T \leq 450^\circ\text{C} \\ f'_c \left[2.011 - 2.353 \left(\frac{T - 20}{1000} \right) \right] & 450^\circ\text{C} < T \leq 874^\circ\text{C} \\ 0 & T > 874^\circ\text{C} \end{cases}$$

$$\varepsilon_{\max,T} = 0.0025 + (6.0T + 0.04T^2) \times 10^{-6}$$

The stress-strain relationship of concrete specified in Eurocode 2 (2004) is

$$\sigma_c = \frac{3\varepsilon f'_{c,T}}{\varepsilon_{c1,T} \left(2 + \left(\frac{\varepsilon}{\varepsilon_{c1,T}} \right)^3 \right)}, \quad \varepsilon \leq \varepsilon_{cu1,T}$$

For $\varepsilon_{cl}(T) < \varepsilon \leq \varepsilon_{cul}(T)$, the Eurocode permits the use of linear as well as nonlinear descending branch in the numerical analysis. For the parameters in this equation refer to Table A.1.

Table A.1 Values for main parameters of the stress-strain relationships
of NSC at elevated temperature (Eurocode 2)

T (°C)	Normal strength concrete					
	Siliceous aggregate			Carbonate aggregate		
	$\frac{f'_{c,T}}{f'_c(20^\circ C)}$	$\varepsilon_{cl,T}$	$\varepsilon_{cul,T}$	$\frac{f'_{c,T}}{f'_c(20^\circ C)}$	$\varepsilon_{cl,T}$	$\varepsilon_{cul,T}$
20	1	0.0025	0.02	1	0.0025	0.02
100	1	0.004	0.0225	1	0.004	0.023
200	0.95	0.005	0.025	0.97	0.0055	0.025
300	0.85	0.007	0.0275	0.91	0.007	0.028
400	0.75	0.01	0.03	0.85	0.01	0.03
500	0.6	0.015	0.0325	0.74	0.015	0.033
600	0.45	0.025	0.035	0.6	0.025	0.035
700	0.3	0.025	0.0375	0.43	0.025	0.038
800	0.15	0.025	0.04	0.27	0.025	0.04
900	0.08	0.025	0.0425	0.15	0.025	0.043
1000	0.04	0.025	0.045	0.06	0.025	0.045
1100	0.01	0.025	0.0475	0.02	0.025	0.048
1200	0	-	-	0	-	-

A.2 Reinforcing steel

A.2.1 Thermal strain

The thermal strain in reinforcing steel specified in ASCE Handbook (Lie 1992) is

$$\varepsilon_{ths} = [0.004(T^2 - 400) + 6(T - 20)] \times 10^{-6}$$

The thermal strain in reinforcing steel specified in Eurocode 2 (2004) is

$$\varepsilon_{ths} = \begin{cases} 1.2 \times 10^{-5} T + 0.4 \times 10^{-8} T^2 - 2.416 \times 10^{-4} & 20^\circ\text{C} \leq T \leq 750^\circ\text{C} \\ 1.1 \times 10^{-2} & 750^\circ\text{C} < T \leq 860^\circ\text{C} \\ 2 \times 10^{-5} T - 6.2 \times 10^{-3} & 20^\circ\text{C} \leq T \leq 750^\circ\text{C} \end{cases}$$

A.2.2 Stress-strain relationship

The stress-strain relationship of reinforcing steel specified in ASCE Handbook (Lie 1992) is

$$\sigma_s = \begin{cases} \frac{f(T, 0.001)}{0.001} \varepsilon_s, & \varepsilon_s \leq \varepsilon_p \\ \frac{f(T, 0.001)}{0.001} \varepsilon_p + f(T, \varepsilon_s - \varepsilon_p + 0.001) - f(T, 0.001), & \varepsilon_s > \varepsilon_p \end{cases}$$

$$f(T, x) = 6.9(50 - 0.04T)[1 - \exp((-30 + 0.03T)\sqrt{x})]$$

$$\varepsilon_p = 4 \times 10^{-6} f_{y,20}$$

where: σ_s and ε_s are stress (MPa) and strain in steel reinforcement, respectively, and $f_{y,20}$ is the yield strength of reinforcing steel (MPa) at room temperature.

The stress-strain relationship of reinforcing steel specified in Eurocode (2004) is

$$\sigma_s = \left\{ \begin{array}{ll} \varepsilon_s E_{s,T} & \varepsilon_s \leq \varepsilon_{sp,T} \\ f_{sp,T} - c + (b/a)(a^2 - (\varepsilon_{sy,T} - \varepsilon_s)^2)^{0.5} & \varepsilon_{sp,T} < \varepsilon_s \leq \varepsilon_{sy,T} \\ f_{sy,T} & \varepsilon_{sy,T} < \varepsilon_s \leq \varepsilon_{st,T} \\ f_{sy,T} \left(1 - \frac{\varepsilon_s - \varepsilon_{st,T}}{\varepsilon_{su,T} - \varepsilon_{st,T}} \right) & \varepsilon_{st,T} < \varepsilon_s \leq \varepsilon_{su,T} \\ 0.0 & \varepsilon_s > \varepsilon_{su,T} \end{array} \right\}$$

$$\varepsilon_{sp,T} = \frac{f_{sp,T}}{E_{s,T}}, \varepsilon_{sy,T} = 0.02, \varepsilon_{st,T} = 0.15, \varepsilon_{su,T} = 0.2$$

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

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

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

The values of $f_{sp,T}$, $f_{sy,T}$, and $E_{s,T}$ can be obtained from Table A.2.

Table A.2 Values for main parameters of stress-strain relationships of reinforcing steel at elevated temperatures (Eurocode 2)

Steel temperature T (°C)	f_{yT}/f_y	f_{sp}/f_y^*	E_{sT}/E_s^*
20	1	1	1
100	1	1	1
200	1	0.807	0.9
300	1	0.613	0.8
400	1	0.42	0.7
500	0.78	0.36	0.6
600	0.4	0.18	0.31
700	0.23	0.075	0.13
800	0.11	0.05	0.09
900	0.06	0.0375	0.0675
1000	0.04	0.025	0.045
1100	0.02	0.0125	0.0225
1200	0	0	0

* f_y and E_s are yield strength and modulus of elasticity at room temperature.

A.3 FRP

A.3.1 Thermal Properties (proposed by Griffis et al. 1984)

A.3.1.1 Specific heat ($c_{w,T}$)

In the following equations, specific heat ($c_{w,T}$) has units of (kJ/kg -°C) and T_w in °C

$$0 \leq T_w \leq 325: c_{w,T} = 1.25 + \frac{0.95}{325} T_w$$

$$325 \leq T_w \leq 343: c_{w,T} = 2.2 + \frac{2.8}{18} (T_w - 325)$$

$$343 \leq T_w \leq 510: c_{w,T} = 5.0 + \frac{-0.15}{167} (T_w - 343)$$

$$510 \leq T_w \leq 538: c_{w,T} = 4.85 + \frac{-3.59}{28} (T_w - 510)$$

$$538 \leq T_w \leq 3316: c_{w,T} = 1.265 + \frac{1.385}{2778} (T_w - 538)$$

$$T_w \geq 3316: c_{w,T} = 0$$

A.3.1.2 Density ($\rho_{w,T}$)

In the following equations, density ($\rho_{w,T}$) has units of (g/cm³) and T_w in °C

$$0 \leq T_w \leq 510: \rho_{w,T} = 1.6$$

$$510 \leq T_w \leq 538: \rho_{w,T} = 1.6 + \frac{-0.35}{28} (T_w - 510)$$

$$538 \leq T_w \leq 120: \rho_{w,T} = 1.25$$

A.3.1.3 Thermal conductivity ($k_{w,T}$)

In the following equations, thermal conductivity ($k_{w,T}$) has units of (W/m-°C) and T_w in °C

$$0 \leq T_w \leq 500: k_{w,T} = 1.4 + \frac{-1.1}{500} T_w$$

$$500 \leq T_w \leq 650: k_{w,T} = 1.4 + \frac{-0.1}{150} (T_w - 500)$$

$$T_w \geq 650: k_{w,T} = 0.2$$

A.3.2 Mechanical Properties

A.3.2.1 Tensile strength and elastic modulus for internal FRP rebar and external FRP laminates (proposed by Bisby et al. 2005)

In the following equations, the units of tensile strength ($f_{f,T}$) and elastic modulus ($E_{f,T}$) are MPa for temperature T_w in °C

$$f_{f,T} = f_{20^\circ C} \left(\frac{1-a_\sigma}{2} \right) \tanh(-b_\sigma(T_w - c_\sigma) + \frac{1+a_\sigma}{2})$$

$$E_{f,T} = E_{20^\circ C} \left(\frac{1-a_E}{2} \right) \tanh(-b_E(T_w - c_E) + \frac{1+a_E}{2})$$

where, $f_{20^\circ C}$ and $E_{20^\circ C}$ are tensile strength and modulus of FRP at ambient conditions.

$$\text{CFRP: } a_\sigma = 0.1, b_\sigma = 5.83e-3, c_\sigma = 339.54$$

$$a_E = 0.05, b_E = 8.68e-3, c_E = 367.41$$

$$\text{GFRP: } a_\sigma = 0.1, b_\sigma = 8.10e-3, c_\sigma = 289.14$$

$$a_E = 0.05, b_E = 7.91e-3, c_E = 320.35$$

$$\text{AFRP: } a_\sigma = 0.1, b_\sigma = 8.48e-3, c_\sigma = 287.65$$

$$a_E = 0.05, b_E = 7.93e-3, c_E = 290.49$$

A.3.2.2 Tensile strength and elastic modulus for NSM FRP reinforcement (proposed by Yu and Kodur 2013)

The following equations provide reduction factors of tensile strength and elastic modulus at elevated temperatures.

For CFRP strip, the reduction factors are

$$\text{Strength: } f(T) = 0.56 - 0.44 \tanh(0.0052(T - 305))$$

$$\text{Modulus: } E(T) = 0.51 - 0.49 \tanh(0.0035(T - 340))$$

For CFRP rod, the reduction factors are

$$\text{Strength: } f(T) = 0.54 - 0.46 \tanh(0.0064(T - 330))$$

$$\text{Modulus: } E(T) = 0.51 - 0.49 \tanh(0.0033(T - 320))$$

where T is FRP temperature in °C, $f(T)$ and $E(T)$ are reduction factors for tensile strength and elastic modulus respectively.

A.3.3 Bond Properties

A.3.3.1 Bond properties of internal FRP rebar

The following equations provide the variation of bond strength for internal FRP rebar at elevated temperatures (proposed by Katz and Berman 2000).

$$\tau = 0.5(1 - \tau_r) \tanh \left\{ -\frac{0.02}{Cr} \left[T - k_1 \left(T_g + \frac{k_1}{0.02} Cr \right) \right] \right\} + 0.5(1 + \tau_r)$$

$$k_1 = \begin{cases} 1, & T_g \leq 80, \\ 1 - 0.025(T_g - 80) & 80 < T_g < 120, \\ 0 & T_g \geq 120 \end{cases}$$

where, τ is the normalized bond strength, T is the temperature, τ_r is the residual bond strength, Cr is the degree of cross-linking, T_g is the glass transition temperature of polymer.

A.3.3.2 Bond properties of external FRP laminates

Ahmed (2010) compiled the available test data on bond degradation in externally bonded FRP, and the following temperature dependent bond strength was proposed.

$$\frac{f_T}{f_{20}} = 1 - \left(\frac{1}{80} \right) (T - 40) \quad (40^\circ\text{C} \leq T \leq 120^\circ\text{C})$$

where, f_{20} and f_T are the bond strength at room and higher temperatures respectively, T is the temperature at the interface of FRP and concrete.

A.3.3.3 Bond properties of NSM FRP reinforcement

The following equations provide reduction factors of bond strength and bond modulus for NSM FRP over a wide temperature range (20-400°C). These relations are based on a number of pull-out tests performed by Yu and Kodur (2013).

For Tyfo T300 epoxy, the reduction factors are

$$\text{Bond strength: } \tau(T) = 0.55 - 0.45 \tanh(0.011(T - 119))$$

$$\text{Bond modulus: } E(T) = 0.59 - 0.41 \tanh(0.01(T - 143))$$

For Tyfo S epoxy, the reduction factors are

$$\text{Bond strength: } \tau(T) = 0.55 - 0.45 \tanh(0.012(T - 129))$$

$$\text{Bond modulus: } E(T) = 0.6 - 0.4 \tanh(0.009(T - 143))$$

where T is NSM epoxy temperature in °C, $\tau(T)$ and $E(T)$ are reduction factors for bond strength and bond modulus respectively.

A.4 NSM Epoxy Adhesive

There are no studies conducted to evaluate thermal properties of NSM epoxy adhesives. However, previous studies on thermal properties of other types of epoxy can provide a rough estimation on the values of thermal properties. Table A.3 presents some of previous studies on thermal properties of epoxy at elevated temperatures.

Table A.3 Previous studies on thermal properties of epoxy

Reference	Material	Thermal conductivity (W/m-°C)	Specific heat (kJ/kg-°C)	Temperature range (°C)
Chern et al. (2002)	Hercules 3501-6 resin	0.18-0.32	1.3-2.3	23-223
Shokralla and Al-Muaikel (2010)	Epoxy phenolic resin	0.134	--	30-100
Kandare et al. (2010)	Epoxy resin	0.16	0-2.3	20-450
NPL (2013)	Epoxy cast resins	0.17-0.21	1.11-2.11	20-200

It can be seen in Table A3 that the values of thermal conductivity and specific heat are reasonably close for different types of epoxy. The values of thermal conductivity vary in the range of 0.13-0.32 W/m-°C, whereas those of specific heat vary in the range of 0-2.3 kJ/kg-°C. Thus, the experimental data reported by Chern et al. (2002) is applied in the numerical model for thermal properties of NSM epoxy adhesives. The temperature dependent thermal conductivity is:

$300\text{K} \leq T_e \leq 500\text{K}$:

$$k_{e,T} = (0.202 + 6.122 \times 10^{-3} T - 4.8107 \times 10^{-5} T^2 + 1.248 \times 10^{-7} T^3 - 1.043 \times 10^{-1} T^4)$$

where, T_e is epoxy temperature in K, $k_{e,T}$ is thermal conductivity of epoxy in W/m-K.

The temperature dependent specific heat is:

$$300\text{K} \geq T_e \geq 502\text{K}: c_{e,T} = (5.34T - 456.9)$$

$$T_e \geq 502\text{K}: c_{e,T} = (2867.6 - 13.322T + 4.304 \times 10^{-2} T^2 - 3.776 \times 10^{-5} T^3)$$

where, $c_{e,T}$ is specific heat of epoxy in kJ/kg-K.

A.5 Insulation

A.5.1 Tyfo Vermiculite-Gypsum (VG)

This insulation is manufactured by FYFE Co. LLC as fire proofing system for FRP composites. These thermal properties relationships are based on therogravimetric analysis (TGA) performed by Bisby (2003).

A.5.1.1 Density

The VG insulation has two primary components, namely, gypsum and vermiculite. Based on typical densities of gypsum (865 kg/m^3) and vermiculite (128 kg/m^3) mixed in 2:1 ratio, the relationships obtained through TGA are:

$$0 \leq T_{VG} \leq 100: \rho_{VG,T} = 351$$

$$100 \leq T_{VG} \leq 200: \rho_{VG,T} = 351 - \frac{351-287}{200-100}(T_{VG} - 100)$$

$$T_{VG} \geq 200: \rho_{VG,T} = 287$$

where, ρ_{VG} is density in kg/m^3 and temperature T_{VG} in $^{\circ}\text{C}$.

A.5.1.2 Specific heat

The two components of insulation (vermiculite and gypsum) have different specific heat values with temperature variation. For specific heat relationships presented below, it has been assumed that specific heat of vermiculites remains constant whereas it changes with temperature for gypsum. The effect of dehydration has been included by artificially increasing the specific heat around 100°C .

$$0 \leq T_{VG} \leq 20: c_{VG,T} = 1.1763$$

$$20 \leq T_{VG} \leq 78: c_{VG,T} = 1.1763 + \frac{1.3058-1.1763}{78-20}(T_{VG} - 20)$$

$$78 \leq T_{VG} \leq 125: c_{VG,T} = 1.3058 + \frac{6.9066-1.3058}{125-78}(T_{VG} - 78)$$

$$125 \leq T_{VG} \leq 137: c_{VG,T} = 6.9066 + \frac{1.3722-1.1763}{137-125}(T_{VG} - 125)$$

$$137 \leq T_{VG} \leq 153: c_{VG,T} = 1.3722 + \frac{1.3722-1.0136}{153-137}(T_{VG} - 137)$$

$$153 \leq T_{VG} \leq 610: c_{VG,T} = 1.0136 + \frac{1.0136-0.8609}{610-153}(T_{VG} - 153)$$

$$610 \leq T_{VG} \leq 663: c_{VG,T} = 0.8509 + \frac{1.6976-0.8509}{663-610}(T_{VG} - 610)$$

$$663 \leq T_{VG} \leq 690: c_{VG,T} = 1.6976 + \frac{1.6976 - 0.9167}{690 - 663}(T_{VG} - 663)$$

$$T_{VG} \geq 690: c_{VG,T} = 0.9167$$

where, c_{VG} is specific heat of VG insulation in J/kg-°C and temperature T_{VG} in °C.

A.5.1.3 Thermal conductivity

The thermal conductivity of vermiculite is constant with temperature and that of gypsum varies with temperature. The variation of thermal conductivity k_{VG} (W/m-°C) with temperature T_{VG} (°C) is expressed by:

$$0 \leq T_{VG} \leq 100: k_{VG,T} = 0.1158$$

$$100 \leq T_{VG} \leq 101: k_{VG,T} = 0.1158 + \frac{0.1158 - 0.0726}{101 - 100}(T_{VG} - 100)$$

$$101 \leq T_{VG} \leq 400: k_{VG,T} = 0.0726$$

$$400 \leq T_{VG} \leq 800: k_{VG,T} = 0.0726 - \frac{0.1224 - 0.0726}{800 - 400}(T_{VG} - 400)$$

$$T_{VG} \geq 800: k_{VG,T} = 0.1224 - \frac{0.2087 - 0.1224}{1000 - 800}(T_{VG} - 800)$$

A.5.2 Promatect Calcium – Silicate Boards

The following thermal properties relationships are based on reported thermal properties in the literature or those specified by manufacture (Deuring 1994, Blontrock et al. 2000, Ahmed 2010).

A.5.2.1 Density

The following values of density of Promatech insulation are proposed by Deuring (1994) and Blontrock et al. (2000).

$$\text{Promatech-H: } \rho_i = 870$$

$$\text{Promatech-100: } \rho_i = 875$$

$$\text{Promatech-L: } \rho_i = 500$$

A.5.2.2 Specific heat

The specific heat (J/kg-°C) for calcium silicate insulating slabs is:

(www.nu-techresources.com/datasheet/PROMATECTH-eng.pdf)

$$\text{Promatech-H: } c_i = 920$$

$$\text{Promatech-100: } c_i = 840$$

$$\text{Promatech-L: } c_i = 950$$

A.5.2.3 Thermal conductivity

The thermal conductivity $k_{i,T}$ of Promatech insulation (J/kg-°C) at various temperatures

T_i (°C) is:

$$\text{Promatech-H: } 0 \leq T_i \leq 390: k_{i,T} = (1.833e - 4)T + 0.175$$

$$T_i \geq 390: k_{i,T} = 0.25$$

$$\text{Promatech-100: } k_{i,T} = 0.285$$

$$\text{Promatech-L: } 0 \leq T_i \leq 100: k_{i,T} = (7.07e-5)T + 0.083$$

$$100 \leq T_i \leq 200: k_{i,T} = (4.0e-5)T + 0.086$$

$$200 \leq T_i \leq 400: k_{i,T} = (6.0e-5)T + 0.082$$

$$400 \leq T_i \leq 500: k_{i,T} = (8.0e-5)T + 0.074$$

$$T_i \geq 500: k_{i,T} = 0.144$$

APPENDIX B

Design and Load Calculations

This Appendix summarizes the design and load calculations on conventional RC beam using ACI 318R (2011) and NSM FRP strengthened RC beam using ACI 440.2R (2008). The cross-section, shear force diagram, and bending moment diagram for the tested beams are shown in Figure B.1. The design calculations are presented in the following two sections.

B.1 Design of RC T-beam

1. Configurations and materials

Configurations: See Figure B.1.

Material properties: $f_c' = 41.4 \text{ MPa}$, $\varepsilon_c = 0.003$, $f_y = 414 \text{ MPa}$

2. Flexural capacity as per ACI-318R (2011)

$$d = 406.4 - 50.8 - 6.4 - 9.5 = 339.7 \text{ mm} \text{ (Clear concrete cover is } 50.8 \text{ mm)}$$

Since Neutral axis is located very close to top rebars, the moment contribution of top layer rebars is neglected.

Based on the force equilibrium principle,

$$\begin{aligned} A_s \cdot f_y &= 0.85 f_c' \cdot b \cdot (\beta_1 \cdot c) \\ 854.8 \text{ mm}^2 \times 414 \text{ MPa} &= 0.85 \times 41.4 \text{ MPa} \times 432 \text{ mm} \times (0.75c) \end{aligned}$$

$$c = 31.1 \text{ mm}$$

Based on the strain compatibility principle,

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_t}{d - c}$$

$$\frac{0.003}{31.1} = \frac{\varepsilon_t}{339.7 - 31.1}$$

$\varepsilon_t = 0.0298 > 414 \text{ MPa} / 201000 \text{ MPa} = 0.0021$, steel yields. The assumption is correct!

Check minimum reinforcement (ACI 318R -10.5.1)

$$\rho_{min} = 0.0039$$

$$\rho = \frac{A_s}{b \cdot d} = \frac{854.8}{432 \times 339.7} = 0.0058 > \rho_{min} \quad O.K.$$

Check minimum reinforcement spacing (ACI 318R - 7.6.1)

$$space = (228.6 - 50.8 - 50.8 - 2 \times 6.4 - 3 \times 19) / 2$$

$$= 28.6 > 25.4 \text{ mm or dia. of rebar} \quad O.K.$$

The moment capacity of the beam is

$$M_n = A_s f_y \cdot \left(d - \frac{a}{2}\right)$$

$$= 854.8 \times 414 \times (339.7 - 0.5 \times 0.75 \times 31.1) / 10^6$$

$$= 116.1 \text{ kN} \cdot \text{m}$$

$$M_n = 1.4 P_n$$

$$P_n = 82.9 \text{ kN and } P_u = 74.6 \text{ kN}$$

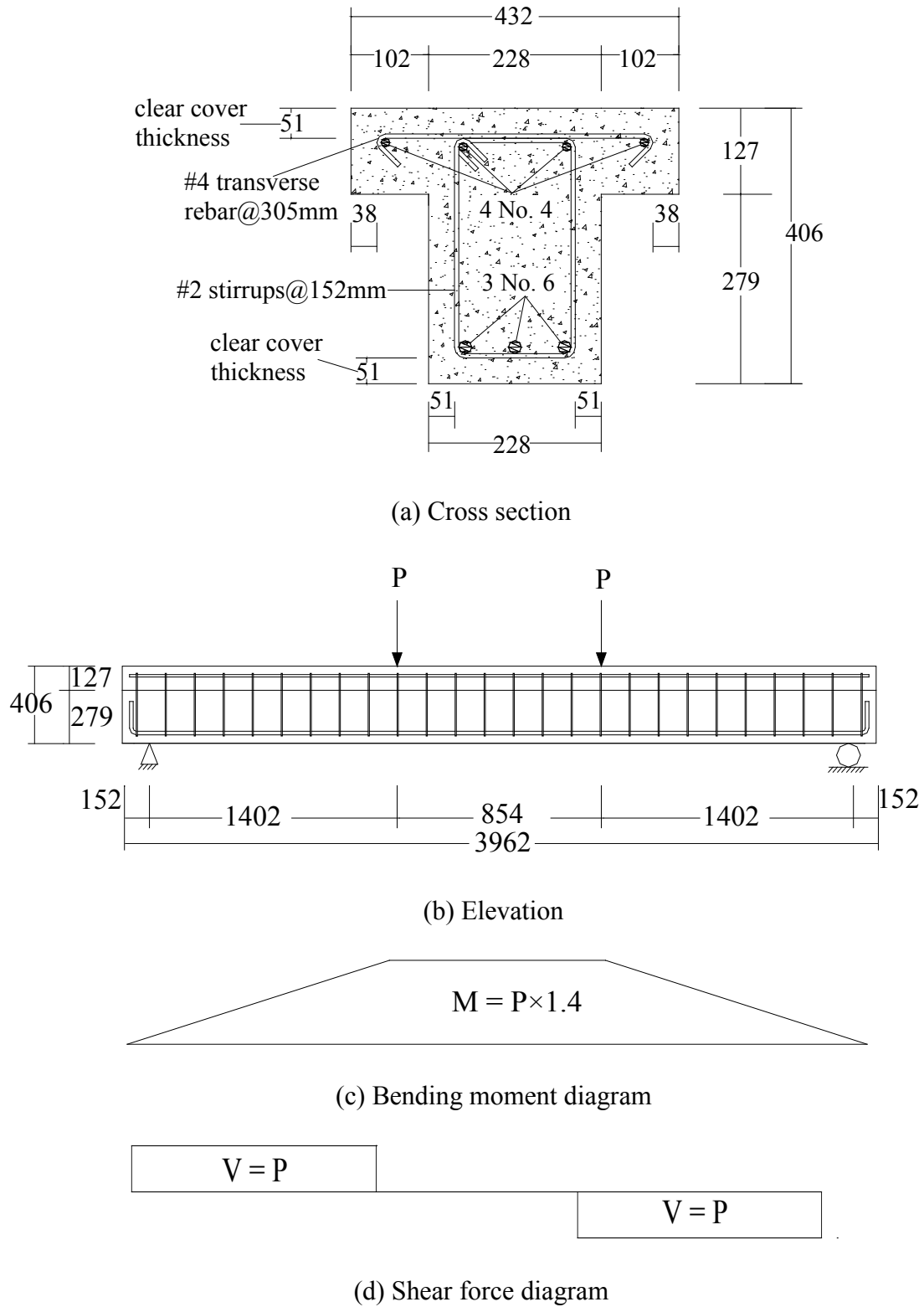


Figure B.1 Cross section, elevation and internal force diagram of RC T-beam

3. Shear capacity as per ACI-318 (#2 stirrups @150mm)

Shear capacity provided by concrete

$$V_c = 0.16\sqrt{f'_c} \cdot b_w \cdot d = 2 \times \sqrt{41.4} \times 228.6 \times 339.7 / 1000 = 79.9kN$$

Shear capacity provided by stirrups

$$V_s = \frac{A_v \cdot f_{yt} \cdot d}{s} = \frac{2 \times 31.6 \times 241 \times 339.7}{150} / 1000 = 34.5kN$$

$$V_n = V_c + V_s = 79.9 + 34.5 = 114.4kN$$

$$\phi V_n = 0.75 \times 114.4 = 85.8kN > P_u = 74.6kN$$

$$\begin{aligned} A_{v,min} &= 0.06\sqrt{f'_c} b_w s / f_{yt} = 0.75 \times \sqrt{41.4} \times 228.6 \times 150 / 241 / 1000 \\ &= 0.009 < 2 \times 0.049 = 0.098 \quad O.K. \end{aligned}$$

Therefore, using #2 stirrups at 150 mm c/c satisfies shear capacity requirement.

4. T beam configuration check

1) Width of flange (ACI 318R-8.12.2)

$$width \leq \frac{1}{4} \times span = \frac{1}{4} \times 3658 = 914.5 \quad O.K.$$

$$overhanging = 102 \leq \left\{ \begin{array}{l} 8 \times t_{slab} = 8 \times 127 = 1016 \\ \frac{1}{2} \times clear \text{ dis. to next slab} \end{array} \right\} \quad O.K.$$

2) Flange thickness (ACI 318R - 8.12.4)

$$thickness = 102 \geq \frac{1}{2} \times width \text{ of web} = \frac{1}{2} \times 229 = 114 \quad O.K.$$

3) Transverse reinforcement shall be spaced not farther apart than 5 times the slab thickness

$$space = 152 < 5 \times \text{slab thickness} = 5 \times 127 = 635 \quad O.K.$$

Use #4 rebar @ 150 mm as transverse reinforcement in flange.

B.2 Design of NSM FRP strengthened RC T-beam

1. Material properties

Configurations: See Figure B.2.

Material properties: $f_c' = 41.4 \text{ MPa}$, $\epsilon_c = 0.003$, $f_y = 414 \text{ MPa}$

$$E_{FRP} = 20250 \text{ ksi},$$

$$\epsilon_{FRP} = C_E K_a \epsilon_{fu} = 0.95 \times 0.7 \times 0.018 = 0.0117 \text{ (ACI 440.2R -08 10.1.1)}$$

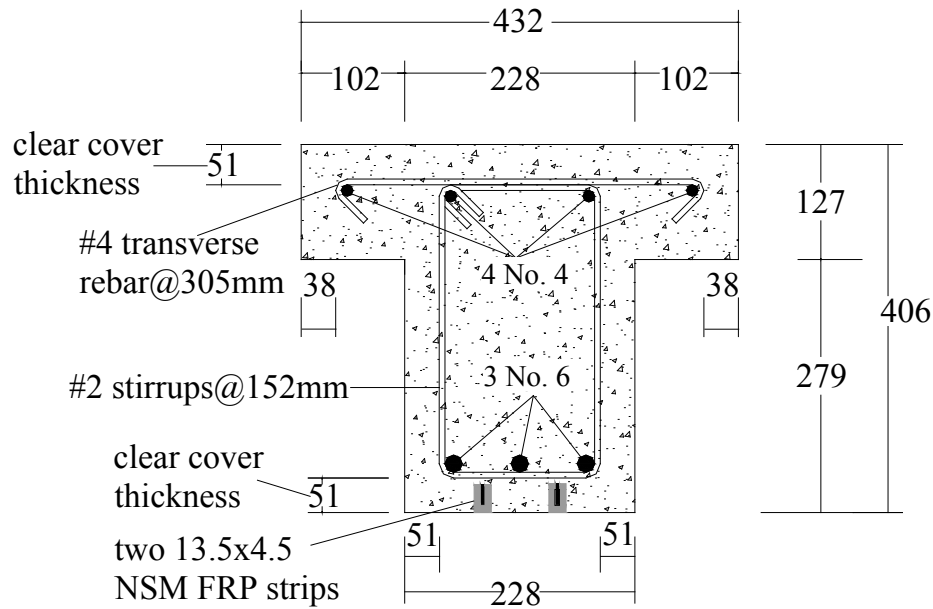


Figure B.2 Configuration of NSM FRP strengthened RC T-beam

2. Flexural capacity as per ACI 440.2R (2008)

Based on the force equilibrium principle,

$$A_s \cdot f_y + A_{FRP} \cdot f_{FRP} = 0.85 f'_c \cdot b \cdot (\beta_1 \cdot c)$$

$$854.8 \text{ mm}^2 \times 414 \text{ MPa} + 121 \text{ mm}^2 \times (0.0117 \times 139.7 \text{ GPa}) = 0.85 \times 41.4 \text{ MPa} \times 432 \text{ mm} \times (0.75c)$$

$$c = 47.4 \text{ mm} < 127 \text{ mm} \text{ (within the flange)}$$

Based on the strain compatibility principle,

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_s}{d_s - c} = \frac{\varepsilon_f}{d_f - c}$$

$$\frac{0.003}{47.4} = \frac{\varepsilon_s}{339.7 - 47.4} = \frac{\varepsilon_f}{387.4 - 47.4}$$

$$\varepsilon_s = 0.0165 > 414 \text{ MPa} / 201000 \text{ MPa} = 0.0021, \text{ steel yields.}$$

$\varepsilon_s = 0.0215 > 0.0117$, FRP fails. Therefore, FRP rupture or debonding controls flexural failure.

The nominal moment capacity of the beam is

$$M_n = A_s f_y \cdot (d_s - \frac{a}{2}) + \psi_f A_{FRP} f_{FRP} \cdot (d_f - \frac{a}{2})$$

$$= 854.8 \times 414 \times (339.7 - 0.5 \times 0.75 \times 47.4) / 10^6$$

$$+ 0.85 \times 121 \times (0.117 \times 139700) \times (387.4 - 0.5 \times 0.75 \times 47.4) / 10^6$$

$$= 175.9 \text{ kN} \cdot \text{m}$$

The increase in moment capacity is 51.5%

$$M_n = 1.4 P_n$$

$$P_n = 125.6 \text{ kN} \text{ and } P_u = 113 \text{ kN}$$

The load ratio is given as:

$$LR = \frac{62}{125.6} \times 100\% = 49.4\%$$

APPENDIX C

Finite Element Formulation

To solve the heat and mass transfer problems, the cross-section of the beam segment is divided into rectangular elements as shown in Figure 5.1. Since the dependent variable (the variable to be computed) in the two problems is scalar, Q4 (quadrilateral element that has four nodes) element is used in the analysis. Due to the nonlinearity of both problems, the integrations in Eqns. (5.11) through (5.13) are evaluated numerically using Gaussian quadrature integration technique. The vector of shape functions for Q4 element can be written as:

$$N = \begin{bmatrix} (1-s)(1-t) / 4 \\ (1+s)(1-t) / 4 \\ (1+s)(1+t) / 4 \\ (1-s)(1+t) / 4 \end{bmatrix} \quad (C.1)$$

where: s and t are transformed coordinates as shown in Figure C.1. The analysis is generally carried out using four Gauss points and the element stiffness matrix (K_e), mass matrix (M_e) and nodal heat or mass flux (F_e) are evaluated at every Gauss point. Those values of the element matrices at the four Gauss points are summed to form the element material property matrices which are used for the subsequent steps in the analysis.

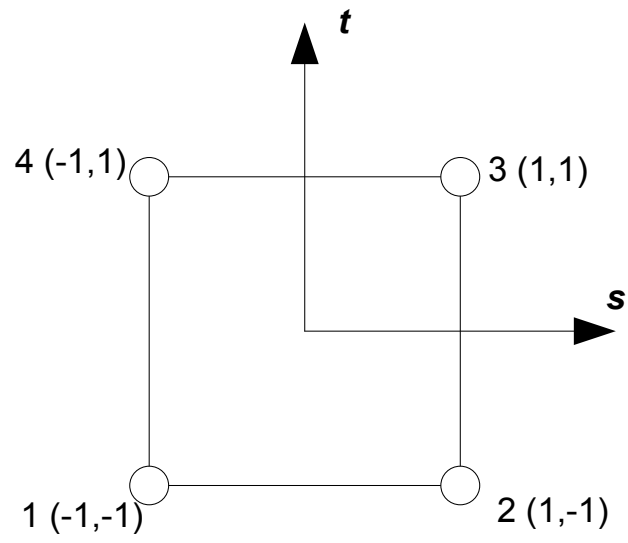


Figure C.1 Q4 element in transformed coordinates

APPENDIX D

Design Examples

D.1 Example 1 - Fire resistance of NSM FRP strengthened RC beams without fire insulation

An NSM FRP strengthened RC beam (carbonate aggregate, normal strength) of span 4.5 m is exposed to a standard ASTM E119 fire and under uniformly distributed load corresponding to 50% of room temperature capacity. The beam size is 200 mm $\times$ 400 mm and the top surface is protected from fire by the concrete floor slab. The beam has three 19 mm diameter reinforcing bars at four corners and the clear cover to the reinforcing bars is 35 mm. The stirrups are of 8 mm dimensions. Two 13.5 $\times$ 4.5 mm CFRP strips are sued as NSM FRP strengthening. Configuration and material properties are tabulated in Table D.1. Calculate moment capacity of fire exposed beam as a function of fire exposure time, and evaluate the fire resistance of beam under such fire and loading conditions.

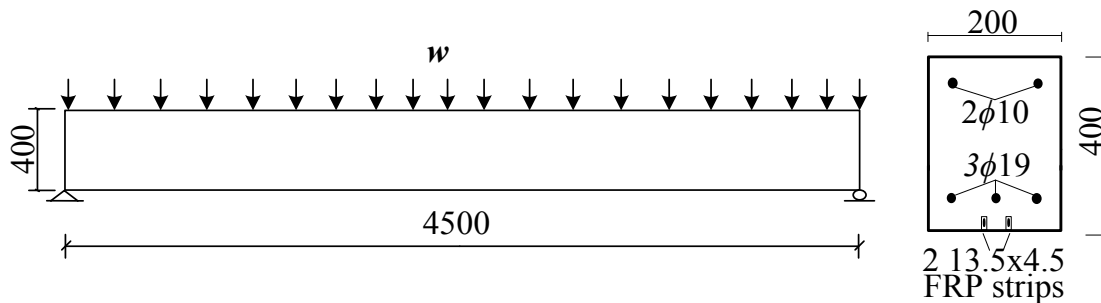


Figure D.1 Layout and cross section of NSM FRP strengthened RC beam (Beam D1)

Table D.1 Properties of Beams D1 and D2

Property		Beam D1	Beam D2
Cross-section (mm)		200×400	250×450
Span (m)		4.5	6
Loading	Magnitude	33.8 kN/m, 50% load ratio	44.3 kN/m, 50% load ratio
	Load type	Uniformly distributed load	Uniformly distributed load
Concrete	Aggregate	Carbonate	Carbonate
	f_c' (MPa)	41.4	41.4
	Cover thickness	35 mm	35 mm
Steel rebar	Top rebar	3 ϕ 19	4 ϕ 19
	Bottom rebar	2 ϕ 10	2 ϕ 12.7
	f_y (MPa)	414	414
FRP strengthening	Strengthening type	NSM	EBR
	Dimension	two 13.5×4.5 mm FRP strips	250×1.0 mm
	f_{FRP} (MPa)	2068	1133
	E_{FRP} (GPa)	131	75.6
	Groove size	25×10 mm	--
VG insulation	Thermal conductivity	No insulation	0.116 W/m-K
	Specific heat		1.176 J/kg-K
	Density		351 kg/m ³

Solutions:

Step 1: Calculate temperatures in steel rebars and NSM FRP strips

Applying proposed 1-D and 2-D heat transfer equations to calculate the temperatures in steel rebar and NSM FRP, respectively.

The distance from the center of corner rebar to two fire exposure side is $(35+8+19/2=53)$ mm, so $y = z = 53$ mm. Take the temperature calculation at 1 hour fire-exposure time as an example. For corner rebar,

$$\eta_z = \eta_y = (0.155 \ln \frac{1}{0.053^{1.5}} - 0.348\sqrt{0.053} - 0.371) = 0.234$$

$$T_c = 1.0 \times (-1.481 \times (0.234 \times 0.234) + 0.985 \times (0.234 + 0.234) + 0.017)(910 \times 1^{0.148}) = 366^\circ C$$

For middle rebar,

$$\eta_z = (0.155 \ln \frac{1}{0.1^{1.5}} - 0.348\sqrt{0.1} - 0.371) = 0.056$$

$$\eta_y = (0.155 \ln \frac{1}{0.053^{1.5}} - 0.348\sqrt{0.053} - 0.371) = 0.234$$

$$T_c = 1.0 \times (-1.481 \times (0.234 \times 0.056) + 0.985 \times (0.234 + 0.056) + 0.017)(910 \times 1^{0.148}) = 261^\circ C$$

For NSM FRP,

$$\eta_z = (0.155 \ln \frac{1}{0.07^{1.5}} - 0.348\sqrt{0.07} - 0.371) = 0.157$$

$$T_c = 1.0 \times (-1.481 \times (0.157 \times 0.565) + 0.985 \times (0.157 + 0.565) + 0.017)(910 \times 1^{0.148}) = 551^\circ C$$

By using spreadsheet the temperatures at other times can be also easily calculated.

The time-temperature curves of rebars and NSM FRP are plotted in Figure D.2.

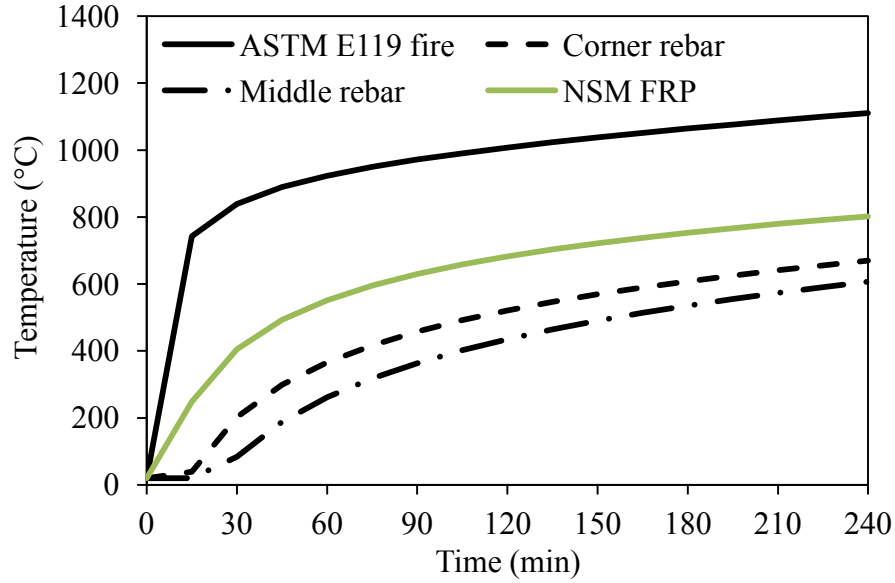


Figure D.2 Variation of temperatures in steel rebar and NSM FRP with fire exposure time in Beam D1

Step 2: Evaluating strength in steel and FRP and effective concrete depth

Knowing the temperature in steel rebars and NSM FRP, the remaining strength in steel rebar can be evaluated using temperature dependent properties specified in Eurocode (2004), and the remaining strength in NSM FRP can be obtained utilizing empirical relations proposed in Chapter 3.

After 1h of fire exposure, the remaining strength in steel is 100% of their room temperature strength for both corner and middle steel rebars, since their temperature is lower than 400°C.

The remaining strength $f(T)$ and modulus $E(T)$ in NSM FRP are

$$\begin{aligned}
 f(T) &= 0.56 - 0.44 \tanh(0.0052(T - 305)) \\
 &= 0.56 - 0.44 \tanh(0.0052(551 - 305)) = 0.183
 \end{aligned}$$

$$E(T) = 0.51 - 0.49 \tanh(0.0035(T - 340))$$

$$= 0.51 - 0.49 \tanh(0.0035(T - 305)) = 0.202$$

To evaluate the effective concrete width at compression zone, Table 7.3 is needed. The effective concrete width for a section of 200×300 can be used in this calculation (conservative). For 1h fire exposure, the effective concrete width is 91.9% of original width.

Step 3: Calculate moment capacity of NSM FRP strengthened beam

Based on the force equilibrium principle,

$$A_s \cdot f_{y,T} + A_{FRP} \cdot f_{FRP,T} = 0.85 f'_c \cdot b_T \cdot (\beta_1 \cdot c)$$

$$854.8 \text{ mm}^2 \times 414 \text{ MPa} + 121 \text{ mm}^2 \times (0.0117 \times 0.202 \times 139.7 \text{ GPa})$$

$$= 0.85 \times 41.4 \text{ MPa} \times 0.919 \times 200 \text{ mm} \times (0.75c)$$

$$c = 73 \text{ mm}$$

Based on the strain compatibility principle,

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_s}{d_s - c} = \frac{\varepsilon_f}{d_f - c}$$

$$\frac{0.003}{73} = \frac{\varepsilon_s}{347 - 73} = \frac{\varepsilon_f}{385 - 73}$$

$$\varepsilon_s = 0.0113 > 414 \text{ MPa} / 201000 \text{ MPa} = 0.0021, \text{ steel yields.}$$

$\varepsilon_s = 0.0128 > 0.0117$, FRP failure governs. Eq. 7.24 can be used. Then the moment capacity of NSM FRP strengthened beam is

$$\begin{aligned}
M_n &= A_s f_y \cdot \left(d_s - \frac{a}{2}\right) + \psi_f A_{FRP} f_{FRP} \cdot \left(d_f - \frac{a}{2}\right) \\
&= 854.8 \times 414 \times (347 - 0.5 \times 0.75 \times 73) / 10^6 \\
&\quad + 0.85 \times 121 \times (0.0117 \times 0.202 \times 139700) \times (385 - 0.5 \times 0.75 \times 73) / 10^6 \\
&= 125.2 \text{ kN} \cdot \text{m}
\end{aligned}$$

By using spreadsheet the moment capacity at other times can also be calculated.

The variation of moment capacity with fire exposure time for NSM FRP strengthened RC beam is plotted in Figure D.3.

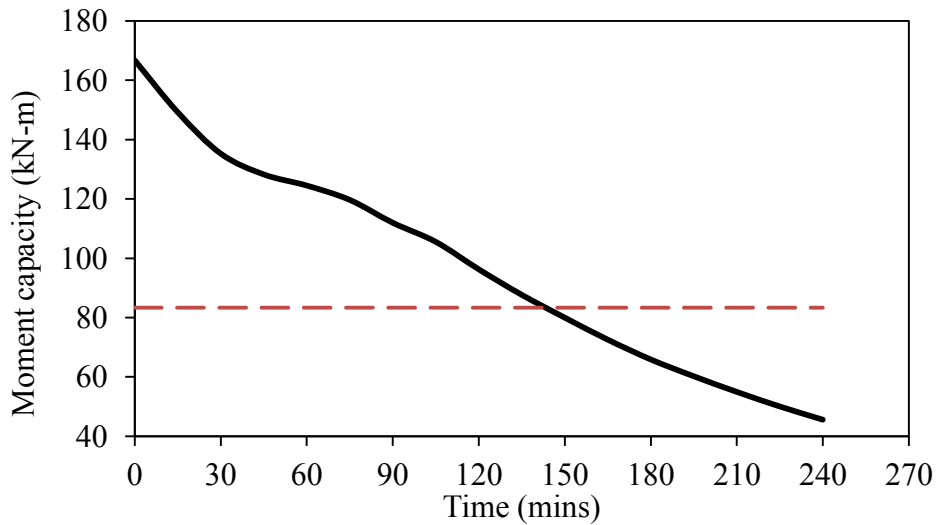


Figure D.3 Variation of moment capacity of Beam D1 with fire exposure time

Step 4: Calculate external load and estimate failure time

The moment capacity of NSM FRP strengthened RC beam at room temperature is calculated to be 168 kN·m as per ACI 440.2 (2008) provisions. Based on the time-moment capacity curve generated in previous step, the beam is estimated to fail at 140 minutes, as shown in Figure D.3. Thus the fire resistance of the beam is 140 minutes.

D.2 Example 2 - Fire resistance of External FRP strengthened RC beams with fire insulation

An external FRP strengthened RC beam (carbonate aggregate, normal strength) of span 6 m is exposed to a standard ASTM E119 fire and under uniformly distributed load corresponding to 50% of room temperature capacity. The beam size is 250 mm $\times$ 450 mm and the top surface is protected from fire by the concrete floor slab. The beam has four 19 mm diameter reinforcing bars at four corners and the clear cover to the reinforcing bars is 38 mm. The stirrups are of 8 mm dimensions. One layer of CFRP laminate (250 $\times$ 1 mm) is used as external FRP strengthening. The beam is protected by U-shaped VG fire insulation with a thickness of 25mm. Configuration and material properties are tabulated in Table D.1. Calculate moment capacity of fire exposed beam as a function of fire exposure time, and evaluate the fire resistance of beam under such fire and loading conditions.

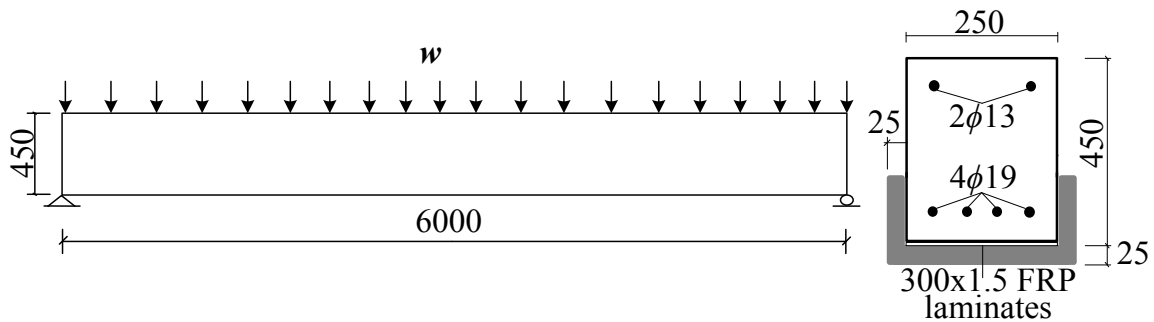


Figure D.4 Layout and cross section of external FRP strengthened RC beam (Beam D2)

Solutions:

Step 1: Calculate temperatures in steel rebars and external FRP laminates

Applying proposed 1-D and 2-D heat transfer equations to calculate the temperatures in steel rebar and external FRP, respectively.

Since the beam is protected by fire insulation, the insulation layer needs to be converted to equivalent concrete layer. It is known that thermal properties of Tyfo VG insulation (Bisby 2003) and concrete (Lie 1992) as follows.

$$k_{VG} = 0.116 \text{ W/m-K}$$

$$(\rho c)_{VG} = 413 \text{ kJ/m}^3\text{-K}$$

$$k_c = 1.355 \text{ W/m-K}$$

$$(\rho c)_c = 2566 \text{ kJ/m}^3\text{-K}$$

Thus, the thickness of equivalent concrete layer is

$$z_{ec} = z_i \sqrt[4]{t} \sqrt[1.75]{\frac{k_c}{(\rho c)_c} \frac{(\rho c)_i}{k_i}} = 25 \times \sqrt[4]{1} \sqrt[1.75]{\frac{1.355}{2.566} \times \frac{0.413}{0.116}} = 36 \text{ mm}$$

The distance from the center of corner rebar to two fire exposure side is $(36+38+8+19/2=92)$ mm, so $y = z = 92$ mm. Take the temperature calculation at 1 hour fire-exposure time as an example. For corner rebar,

$$T_c = 1.0 \times (-1.481 \times (0.08 \times 0.08) + 0.985 \times (0.08 + 0.08) + 0.017)(910 \times 1^{0.148}) = 150^\circ\text{C}$$

For middle rebar,

$$z_{ec} = z_i \sqrt[4]{t} \sqrt[1.75]{\frac{k_c}{(\rho c)_c} \frac{(\rho c)_i}{k_i}} = 25 \times \sqrt[4]{1} \sqrt[1.75]{\frac{1.355}{2.566} \times \frac{0.413}{0.116}} = 36 \text{ mm}$$

$$\eta_z = (0.155 \ln \frac{1}{0.138^{1.5}} - 0.348 \sqrt{0.138} - 0.371) = -0.04$$

$$T_c = 1.0 \times (-1.481 \times (-0.04 \times 0.08) + 0.985 \times (-0.04 + 0.08) + 0.017)(910 \times 1^{0.148}) = 57^\circ\text{C}$$

For external FRP (average temperature of FRP laminates is evaluated use the point at quarter FRP length from fire exposed surface),

$$\eta_z = (0.155 \ln \frac{1}{0.0985^{1.5}} - 0.348 \sqrt{0.0985} - 0.371) = 0.06$$

$$T_c = 1.0 \times (-1.481 \times (0.06 \times 0.33) + 0.985 \times (0.06 + 0.33) + 0.017)(910 \times 1^{0.148}) = 339^\circ\text{C}$$

By using spreadsheet the temperatures at other times can be also calculated. The time-temperatures curve of rebars and external FRP are plotted in Figure D.5.

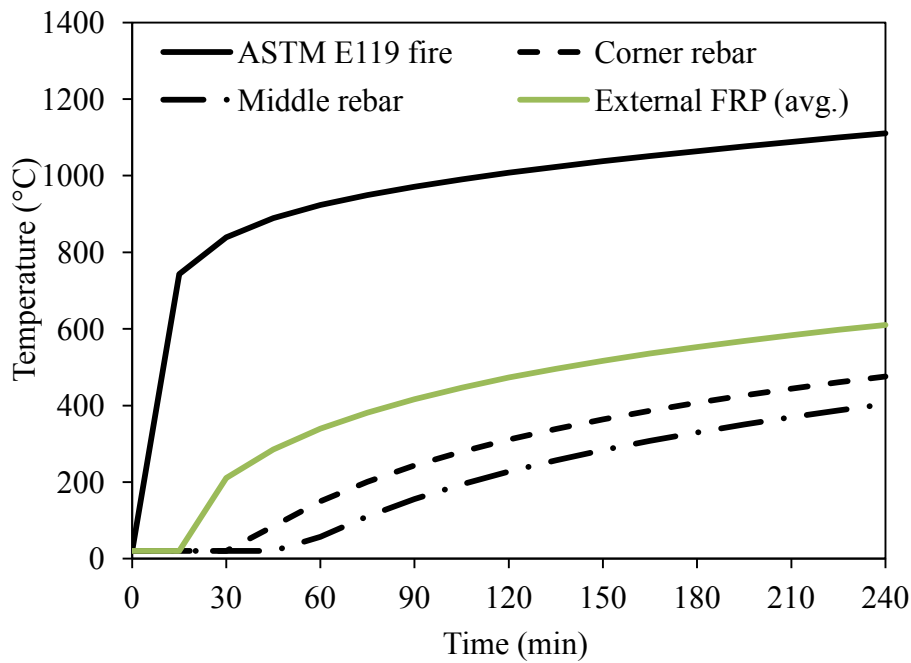


Figure D.5 Variation of temperatures in steel rebar and external FRP with fire exposure time in Beam D2

Step 2: Evaluating the strength in steel and FRP and effective concrete depth

Knowing the temperature in steel rebars and external FRP, the remaining strength in steel rebar can be evaluated using temperature dependent properties specified in

Eurocode (2004), and the remaining strength in external FRP can be obtained utilizing empirical relations proposed by Bisby et al. (2005).

After 1h of fire exposure, the remaining strength in steel is 100% of their room temperature strength for both corner and middle steel rebars, since their temperature is lower than 400°C.

The remaining strength $f(T)$ and modulus $E(T)$ in external FRP are

$$\begin{aligned} f(T) &= 0.55 - 0.45 \tanh(0.00583(T - 339.54)) \\ &= 0.55 - 0.45 \tanh(0.00583(339 - 339.54)) = 0.55 \\ E(T) &= 0.525 - 0.475 \tanh(0.00868(T - 367.41)) \\ &= 0.525 - 0.475 \tanh(0.00868(339 - 367.41)) = 0.64 \end{aligned}$$

To evaluate the effective concrete width at compression zone, Table 7.3 is needed. The effective concrete width for a section of 250×400 can be used in this calculation. For 1h fire exposure, the effective concrete width is 92.8% of original width.

Step 3: Calculate moment capacity of external FRP strengthened beam

Based on the force equilibrium principle,

$$\begin{aligned} A_s \cdot f_{y,T} + A_{FRP} \cdot f_{FRP,T} &= 0.85 f'_c \cdot b_T \cdot (\beta_1 \cdot c) \\ 1134 \text{mm}^2 \times 414 \text{MPa} + 250 \text{mm}^2 \times (0.0045 \times 0.64 \times 75600 \text{MPa}) \\ &= 0.85 \times 41.4 \text{MPa} \times 0.928 \times 250 \text{mm} \times (0.75c) \\ c &= 85.6 \text{ mm} \end{aligned}$$

Effective strain of FRP is

$$\varepsilon_{fd} = 0.41 \sqrt{\frac{f'_c}{nE_f t_f}} \varepsilon_{fu} = 0.41 \sqrt{\frac{41.4}{75600 \times 1.0}} \times 0.015 = 0.0045$$

Based on the strain compatibility principle,

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_s}{d_s - c} = \frac{\varepsilon_f}{d_f - c}$$

$$\frac{0.003}{85.6} = \frac{\varepsilon_s}{394 - 85.6} = \frac{\varepsilon_f}{450.5 - 85.6}$$

$\varepsilon_s = 0.0108 > 414 \text{ MPa} / 201000 \text{ MPa} = 0.0021$, steel yields.

$\varepsilon_s = 0.0128 > 0.0045$, FRP failure governs. Eq. 7.24 can be used. Then the moment capacity of NSM FRP strengthened beam is

$$\begin{aligned} M_n &= A_s f_y \cdot (d_s - \frac{a}{2}) + \psi_f A_{FRP} f_{FRP} \cdot (d_f - \frac{a}{2}) \\ &= 1134 \times 414 \times (394 - 0.5 \times 0.75 \times 85.6) / 10^6 \\ &\quad + 0.85 \times 250 \times (0.0045 \times 0.64 \times 75600) \times (450.5 - 0.5 \times 0.75 \times 85.6) / 10^6 \\ &= 189.3 \text{ kN} \cdot \text{m} \end{aligned}$$

By using spreadsheet the moment capacity at other times can also be calculated.

The variation of moment capacity with fire exposure time for external FRP strengthened RC beam is plotted in Figure D.6.

Step 4: Calculate external load and estimate failure time

The moment capacity of external FRP strengthened RC beam is calculated to be 200 kN·m as per ACI 440.2 (2008) provisions. Based on the time-moment capacity curve generated in previous step, the beam does not fail for four hours of fire exposure. Thus the fire resistance of the beam is at least four hours.

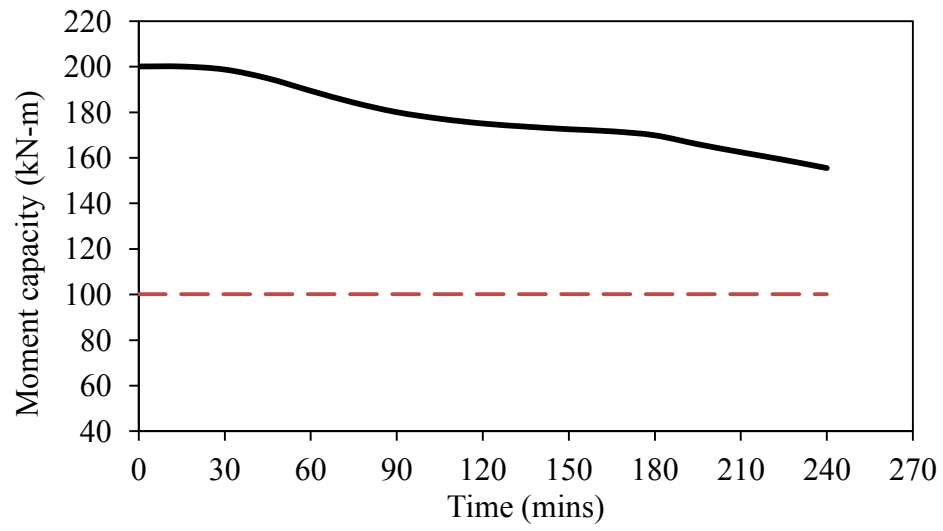


Figure D.6 Variation of moment capacity of Beam D2 with fire exposure time

REFERENCES

REFERENCES

1. Abbasi, A. and Hogg, P.J. (2005) "Prediction of the failure time of glass fiber reinforced plastic reinforced concrete beams under fire conditions." *Journal of Composites for Construction, ASCE*, 9(5), 450-457.
2. Abbasi, A. and Hogg, P.J. (2006) "Fire testing of concrete beams with fiber reinforced plastic rebar." *Composite: Part A*, 37(8), 1142-1150.
3. Anderberg, Y. and Thelandersson, S. (1976) "Stress and deformation characteristics of concrete at high temperatures – Experimental investigation and material behavior model." *Lund Institute of Technology*, Sweden.
4. Ahmed, A. (2010) "Behavior of FRP Strengthened Reinforced Concrete Beams under Fire Conditions." Doctoral Thesis, Michigan State University, East Lansing, MI, USA.
5. Ahmed, A. and Kodur, V. (2011) "Effect of bond degradation on fire resistance of FRP-strengthened reinforced concrete beams." *Composites Part B*, 42(2), 226-237.
6. Ahmed, A. and Kodur, V. (2011) "The experimental behavior of FRP-strengthened RC beams subjected to design fire exposure." *Engineering Structures*, 33(7), 2201-2211.
7. Ali, F.A., Connolly, R.J. and Sullivan, P.J.E. (1996) "Spalling of high strength concrete at elevated temperature." *Applied Fire Science*, 6(3), 3-14.
8. Alkhrdaji, T., Nanni, A., Chen, G. and Barker, M. (1999) "Upgrading the transportation infrastructure: solid RC decks strengthened with FRP." *Concrete International*, 21(10), 37-41.
9. Al-Mahmoud, F., Castel, A., Francois, R. and Tourneur, C. (2011) "Anchorage and tension-stiffening effect between near-surface-mounted CFRP rods and concrete." *Cement and Concrete Composites*, 33(2), 346-352.
10. American Concrete Institute Technical Committee 216. (2007) "Code requirements for determining fire resistance of concrete and masonry construction assemblies." *ACI 216R-07*, Farmington Hills, MI.

11. American Concrete Institute Technical Committee 318. (2011) "Building code requirements for structural concrete and commentary." *ACI 318R-11*, Farmington Hills, MI.
12. American Concrete Institute Technical Committee 440. (2006) "Guide for the design and construction of concrete reinforced with FRP bars." *ACI 440.1R-06*, Farmington Hills, MI.
13. American Concrete Institute Technical Committee 440. (2008) "Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures." *ACI 440.2R-08*, Farmington Hills, MI.
14. American Concrete Institute Technical Committee 440. (2012) "Guide test methods for Fiber-Reinforced Polymers (FRPs) for reinforcing or strengthening concrete structures." *ACI 440.3R-12*, Farmington Hills, MI.
15. American Society of Civil Engineers (ASCE). (2013) "Report Card for America's Infrastructure." Reston, VA.
16. American Society for Testing and Materials. (2012) "Standard test methods for fire tests of building construction and materials." *ASTM E119*, West Conshohocken, PA.
17. American Society for Testing and Materials. (2013) "Standard Test Method for Surface Flammability of Materials Using a Radiant Heat Energy Source." *ASTM E162*, West Conshohocken, PA.
18. American Society for Testing and Materials. (2013) "Standard Test Method for Specific Optical Density of Smoke Generated by Solid Materials." *ASTM E662*, West Conshohocken, PA.
19. American Society for Testing and Materials. (2013) "Standard Test Method for Surface Burning Characteristics of Building Materials." *ASTM E84*, West Conshohocken, PA.
20. American Society for Testing and Materials. (2012) "Standard test method for linear thermal expansion of solid materials by thermomechanical analysis." *ASTM E831*, West Conshohocken, PA.
21. Anagnostopoulos, G., Parthenios, J. and Galiotis, C. (2008) "Thermal stress development in fibrous composites." *Materials Letters*, 62(3), 341-345.
22. Ashby, M.F. and Jones, D.R.H. (1999) *Engineering materials: an introduction to microstructures, processing and design*. Oxford, Pergamon Press.

23. Asplund, S.O. (1949) "Strengthening bridge slabs with grouted reinforcement." *ACI Structural Journal*, 20(4), 397–406.
24. Bank, L.C. (1993) "Properties of FRP reinforcement for concrete," *Fiber-Reinforced-Plastic (FRP) Reinforcement for Concrete Structures: Properties and Applications, Developments in Civil Engineering*, V.42, Elsevier, Amsterdam, 59-86.
25. Barros, J.A.O., Dias, S.J.E. and Lima, J.L.T. (2007) "Efficacy of CFRP-based techniques for the flexural and shear strengthening of concrete beams." *Cement and Concrete Composites*, 29(3), 203-217.
26. Bilotta, A., Ceroni, F., Di Ludovico, M., Nigro, E., Pecce, M. and Manfredi, G. (2011) "Bond efficiency of EBR and NSM FRP systems for strengthening concrete members." *Journal of Composite for Construction*, 15(5), 757-772.
27. Bisby, L.A. (2003) "Fire behavior of fibre-reinforced polymer (FRP) reinforced or confined concrete." Doctoral Thesis, Queen's University, Kingston, Canada.
28. Bisby, L.A., Green, M.F. and Kodur V. (2005) "Response to fire of concrete structures that incorporate FRP." *Progress in Structural Engineering and Materials*, 7(3), 136-149.
29. Blaschko, M. (2003) "Bond behavior of CFRP strips glued into slits." *Proceedings of 6th International Symposium on Fibre-Reinforced Polymer (FRP) Reinforcement for Concrete Structures (FRPRCS-6)*, Singapore, 205-214.
30. Blontrock, H., Taerwe, L., and Vandeveld, P. (2000). "Fire tests on concrete beams strengthened with fibre composite laminates." *Third PhD Symposium*, Vienna, Austria.
31. Blontrock, H., Taerwe, L., and Vanwalleghem, H. (2002). "Bond testing of externally glued FRP laminates at elevated temperature." *Proceeding of the International Conference Bond in Concrete – from research to standards*, Budapest, Hungary, 648-654.
32. BNZ Materials (1998) "Structural Insulation: Summary Data." <http://www.bnzmaterils.com/datasi.html> (Accessed on Feb. 1, 2013)
33. Bowles, D.E. and Tompkins, S.S. (1989) "Prediction of coefficients of thermal expansion for unidirectional composites." *Journal of Composite Materials*, 23(4), 370-388.
34. British Standard Institution. (1987) "Fire tests on building materials and structures - Part 20: method from determination of the fire resistance of elements of construction (general principles)." *BS476-20*, London, UK.

35. Bruggeling, A.S.G. (1992) External prestressing – A state of the Art. *External Prestressing, ACI SP-120*, Detroit, Michigan, 61-82.
36. Buchanan, A. H. (2002). *Structural design for fire safety*, Wiley, Chichester, UK.
37. Burke, P.J., Bisby, L.A. and Green, M.F. (2013) “Effects of elevated temperature on near surface mounted and externally bonded FRP strengthening systems for concrete.” *Cement & concrete composites*, 35(1), 190-199.
38. Campbell, T.I. and Kodur, V.K.R. (1990) “Deformation controlled nonlinear analysis of prestressed concrete continuous beams.” *PCI Journal*, 35(5), 42-55.
39. Canadian Standards Association (CSA) S806-02 (2002) “Design and construction of building components with fibre-reinforced polymers.” *CAN/CSA S806*, Rexdale, Canada.
40. Chang, Y.F., Chen, Y.H., Sheu, M.S. and Yao, G.C. (2006) “Residual stress-strain relationship for concrete after exposure to high temperatures.” *Cement and Concrete Research*, 36(10), 1999-2005.
41. Cheng F.P., Kodur, V.K.R. and Wang, T.C. (2005) “Stress-strain curves for high strength concrete at elevated temperatures”, *Journal of Materials in Civil Engineering*, 16(1), 84-94.
42. Chern, B.C., Moon, T.J., Howell, J.R., and Tan, W. (2002). “New experimental data for enthalpy of reaction and temperature- and degree-of-cure-dependent specific heat and thermal conductivity of the Hercules 3501-6 epoxy system.” *Journal of Composite Materials*, 36(17), 2061-2072.
43. Deuring, M. (1994) “Brandversuche an nachtraglich verstärkten tragern aus beton.” Swiss Federal Laboratories for Materials Testing and Research, Dubendorf, Switzerland.
44. Dotreppe, J.C. and Franssenm, J.M. (1985) “The use of numerical models for the fire analysis of reinforced concrete and composite structures.” *Engineering Analysis*, 2(2), 67-74.
45. Dwaikat, M.M.S. and Kodur, V.K.R. (2013) “A simplified approach for predicting temperatures in fire exposed steel members.” *Fire Safety Journal*, 55(1), 87-96.
46. Dwaikat, M. (2009). “Flexural response of reinforced concrete beams exposed to fire.” Doctoral Thesis, Michigan State University, East Lansing, MI.

47. Dwaikat, M. and Kodur, V.K.R. (2008). "A numerical approach for modeling the fire induced restraint effects in reinforced concrete beams." *Fire Safety Journal*, 43(4), 291-307.
48. Dwaikat, M. and Kodur, V.K.R. (2009). "Response of Restrained Concrete Beams under Design Fire Exposure." *Journal of Structural Engineering, ASCE*, 135(11), 1408-1417.
49. De Lorenzis, L. and Nanni, A. (2002) Bond between NSM fiber-reinforced polymer rods and concrete in structural strengthening. *ACI Structural Journal*, 99(2), 123-32.
50. De Lorenzis, L., Rizzon, A. and La Tegola, A. (2002) "A modified pull-out test for bond of near-surface mounted FRP rods in concrete." *Composites: Part B*, 33(8), 589-603.
51. De Lorenzis, L. and Teng, J.G. (2007) "Near-surface mounted FRP reinforcement - an emerging technique for strengthening structures." *Composites Part B*, 38(2), 119-143.
52. Di Tommaso, A., Neubauer, U., Pantuso, A., and Rostasy, F. S. (2001). "Behaviour of adhesively bonded concrete-CFRP joints at low and high temperatures." *Mechanics of Composite Materials*, 37(4), 327-338.
53. El-Hacha, R and Rizkalla, S.H. (2004) "Near-surface-mounted fiber-reinforced polymer reinforcements for flexural strengthening of concrete structures." *ACI Structural Journal*, 101(5), 717-726.
54. European Committee for Standardization. (2002) "EN 1991-1-2: Actions on structures. Part 1-2: General actions - Actions on structures exposed to fire." *Eurocode 1*, Brussels, Belgium.
55. European Committee for Standardization. (2004) "EN 1992-1-2: Design of concrete structures. Part 1-2: General rules - structural fire design." *Eurocode 2*, Brussels, Belgium.
56. Evseeva, L.E., Tanaeva, S.A. Dubkova, V.I. and Maevskaya, O.I. (2003) "Influence of thermocycling on the thermophysical properties of epoxy compositions hardened with phosphorous carbon fibers." *Journal of Engineering Physics and Thermophysics*, 76(1), 222-225.
57. Fédération Internationale du Béton. (2007) "FRP reinforcement in RC structures." *FIB Bulletin 40*, Lausanne, Switzerland.

58. Firmo, J.P., Correia, J.R. and Franca, P. (2012) "Fire behaviour of reinforced concrete beams strengthened with CFRP laminates: Protection systems with insulation of the anchorage zones." *Composites: Part B*, 43(3), 1545-1556.
59. Flynn, D.R. (1999) "Response of High Performance Concrete to fire conditions: Review of thermal property data and measurement techniques", *NIST GCR 99-767*, MetSys Corporation, U.S.
60. Foye, R. L. (1975) "Creep analysis of Laminates." *Composite Reliability, ASTM STP-580*, 381-395.
61. Franssen J.M., Kodur V.R. and Mason J. (2005) User's manual for SAFIR 2004: A computer program for analysis of structures subjected to fire, University of Liege.
62. Fujisaki, T., Nakatsuji, T. and Sugita, M. (1993) "Research and development of grid shaped FRP reinforcement." *International Symposium on Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, Detroit, Michigan, Mar. 28-31, 177-192.
63. FYFE (2012). "Tyfo® WR-AFP for the Tyfo® CFP system." (Accessed on Feb. 1, 2013) <http://www.fyfeco.com/products/pdf/tyfo%20wr-afp.pdf>
64. Gentry, T.R. and Hudak, C.E.(1996) "Thermal compatibility of plastic composite reinforcements and concrete." *Proceedings of 2nd International Conference on Advanced Composite Materials in Bridges and Structures*, Montreal, Canada, 149-156.
65. Gorji, M. and Mirzadeh, F. (1989) "Theoretical prediction of the thermoelastic properties and thermal stress in unidirectional composites." *Journal of Reinforced Plastics and Composites*, 8(3), 232-258.
66. Griffis, C.A., Masumura, R.A. and Chang, C.I. (1984) "Thermal response of graphite epoxy composite subjected to rapid heating." *Environmental Effects on Composite Materials*, Vol. 2, Technomic Publishing Company, Lancaster, Pennsylvania, 245-260.
67. Harmathy, T.Z. (1967). "A comprehensive creep model." *Journal of Basic Engineering*, 89(3), 496-502.
68. Harmathy, T.Z. (1993). *Fire safety design and concrete*, John Wiley & Sons, Inc., New York, NY.
69. Hassan, T. and Rizkalla, S. (2002). "Flexural strengthening of prestressed bridge slabs with FRP systems." *PCI Journal*, 47(1), 76-93.

70. Hawileh, R.A, Naser, M., Zaidan, W. and Rasheed, H.A. (2009) "Modeling of insulated CFRP-strengthened reinforced concrete T-beam exposed to fire." *Engineering Structures*, 31(12), 3072-3079.
71. Hawileh, R.A, Naser, M. (2012) "Thermal-stress analysis of RC beams reinforced with GFRP rebars." *Composites: Part B*, 43(5), 2135-2142.
72. Hertz, K. (1981) "Simple temperature calculations of fire-exposed concrete constructions." Report No. 159, Institute of Building Design, Technical University of Denmark.
73. Hughes Brothers (2011) <http://aslanfrp.com/Aslan200/Resources/Aslan200.pdf> (accessed Feb. 1, 2013)
74. Intelligent Sensing for Innovative Structures (ISIS) (2001) "Strengthening reinforced concrete structures with externally-bonded fibre reinforced polymers." *Design Manual No. 4*, Winnipeg, Canada.
75. International Organization for Standardization (1999) "Plastics – Thermomechanical analysis (TMA) – Part 2: Determination of coefficient of linear thermal expansion and glass transitions temperature." *ISO 11359-2*, Geneva, Switzerland.
76. International Organization for Standardization (2012) "Fire resistance tests – elements of building construction." *ISO 834*, Geneva, Switzerland.
77. Ishikawa, T., Koyama, K. and Kobayashi, S. (1978) "Thermal expansion coefficient of unidirectional composites." *Journal of Composite Material*, 12(1), 153-168.
78. Isolatek (n.d.) "Spray-Applied Fire Resistive Materials." (Accessed on Feb. 1, 2013) http://www.isolatek.com/category_search.asp?categoryID=1
79. Katz, A., Berman, N. and Bank, L.C. (1999) "Effect of high temperature on bond strength of FRP rebars." *Journal of Composites for Constructions*, 3(2), 73-81.
80. Katz, A. and Berman, N. (2000) "Modelling the effect of high temperature on the bond of FRP reinforcing bars to concrete." *Cement and Concrete Composites*, 22(6), 433-443.
81. Kalagiannakis, G. and Van Hemdriek, D. (2003). "Numerical study on the nonlinear effects of heat diffusion in composite and its potential in non-destructive testing." *Review of Scientific Instruments*, 94(1), 464-465.
82. Kandare, E., Kandola, B.K., Myler, P. and Edwards, G. (2010) "Thermo-mechanical responses of fiber-reinforced epoxy composites exposed to high

- temperature environments. Part I: Experimental data acquisition.” *Journal of Composite Materials*, 44(26), 3093-3114.
83. Khoury, G.A. (1996) “Performance of heated concrete-mechanical properties”, *Contract NUC/56/3604A with Nuclear Installations Inspectorate*, Imperial College, London.
 84. Khoury, G.A. (2000) “Effect of fire on concrete and concrete structures,” *Progress in structural Engineering and materials*, 2(4), 429-447.
 85. Kia, H.G. (1988) “Thermal expansion of polyurethane reinforced with continuous glass fibers.” *Polymer Composites*, 9(3), 237-241.
 86. Klammer, E. L., Hordijk, D. A., and Janssen, H. J. M. (2005) “The influence of temperature on the debonding of externally bonded CFRP.” *Proceedings of 7th International Symposium on Fiber Reinforced Polymer (FRP) Reinforcement for Concrete Structures (FRP7RCS)*, Kansas City, 1551-1592.
 87. Kodur, V.K.R. and Ahmed, A. (2010) “Numerical model for tracing the response of FRP-strengthened RC beams exposed to fire.” *Journal of Composites for Construction*, 14(6), 730-742.
 88. Kodur, V.K.R. and Bisby, L.A. (2005) “Evaluation of fire endurance of concrete slabs reinforced with fiber-reinforced polymer bars.” *Journal of Structural Engineering*, 131(1), 34-43.
 89. Kodur, V.K.R., Bisby, L.A. and Green, M.F. (2006) “Experimental evaluation of the fire behavior of insulated fiber-reinforced-polymer strengthened reinforced concrete columns.” *Fire Safety Journal*, 41(7), 547-557.
 90. Kodur, V. K. R. and Dwaikat, M. (2008). “A numerical model for predicting the fire resistance of reinforced concrete beams.” *Cement and Concrete Composites*, 30(5), 431-443.
 91. Kodur, V.K.R., Dwaikat, M.M.S. and Dwaikat, M.B. (2008) “High-temperature properties of concrete for fire resistance modeling of structures.” *ACI Materials Journal*, 105(5), 517-527.
 92. Kodur, V.K.R., Dwaikat, M. and Raut, N. (2009) “Macroscopic FE model for tracing the fire response of reinforced concrete structures.” *Engineering Structures*, 31(10), 2368-2379.
 93. Kodur, V.K.R. and Harmathy, T.Z. (2008) “Properties of building materials.” *SFPE Handbook of Fire Protections Engineering*, 4th edition, National Fire Protection Association, Quincy, MA, 1167-1195.

94. Kodur, V.K.R. and Phan, L. (2007) "Critical factors governing the fire performance of high strength concrete systems." *Fire Safety Journal*, 42(6), 482-488.
95. Kodur, V.K.R. and Khaliq, W. (2011) "Effect of temperature on thermal properties of different types of high-strength concrete." *Journal of Materials in Civil Engineering*, 23(6), 793-801.
96. Kodur, V.K.R., Wang, T.C., and Cheng, F.P. (2004) "Predicting the fire resistance behavior of high strength concrete columns." *Cement and Concrete Composites*, 26(2), 141-153.
97. Kodur, V.K.R. and Yu, B. (2013) "Evaluating the fire response of concrete beams strengthened with near-surface-mounted FRP reinforcement." *Journal of Composites for Construction, ASCE*, 17(4), 517-529.
98. Kodur, V.K.R., Yu, B. and Dwaikat, M.M.S. (2013) "A simplified approach for predicting temperature in reinforced concrete members exposed to standard fire." *Fire Safety Journal*, Vol. 56, 39-51.
99. Kumahara, S., Masuda, Y., Tanano, H. and Shimizu, A. (1993). "Tensile strength of continuous fiber bar under high temperature." *International Symposium on Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, Detroit, Michigan, Mar. 28-31, 731-742.
100. Kumar, A. (2003) "Behavior of RCC beams after exposure to elevated temperatures" *Journal of the Institution of Engineers (India): Civil Engineering Division*, 84(3), 165-170.
101. Lau, A. and Anson M. (2006) "Effect of high temperatures on high performance steel fiber reinforced concrete", *Cement and Concrete Research*, 36(9), 1698-1707.
102. Leone, M., Matthys, S., and Aiello, M. A. (2009). "Effect of elevated service temperature on bond between FRP EBR systems and concrete." *Composites Part B*, 40(1), 85-93.
103. Lie, T.T. (1992) "Structural fire protection." *ASCE Manuals and Reports of Engineering Practice* 78, New York, NY.
104. Lie, T.T. and Irwin, R.J. (1993) "Method to calculate the fire resistance of reinforced concrete columns with rectangular cross section." *ACI Structural Journal*, 90(1), 52-60.

105. Lie, T.T. and Kodur V.K.R. (1996) "Thermal and mechanical properties of steel-fiber-reinforced concrete at elevated temperatures," *Canadian Journal of Civil Engineering*, 23(2), 511-517.
106. Lin, T.D., Gustaferoo, A.H. and Abrams, M.S. (1981) "Fire endurance of continuous reinforced concrete beams." *Research and Development Bulletin: RD072.01B*. Portland Cement Association, USA.
107. Lu, X., Teng, J., Ye, L., and Jiang, J. (2007). "Intermediate crack debonding in FRP-strengthened RC beams: FE analysis and strength model." *Journal of Composite for Construction, Special Issue: Recent International Advancements in FRP Research and Application in Construction*, 161–174.
108. Manzello, S.L., Park, S.H., Mizukami, T. and Bentz, D.P. (2008) "Measurement of thermal properties of gypsum board at elevated temperatures." *Proceedings of 5th International Conference on Structures in Fire (SIF)*, Singapore, 656-665.
109. Meacham B.J. and Custer R.L.P. (1992) "Performance-based fire safety engineering: an introduction of basic concepts." *Journal of Fire Protection Engineering*, 7(2), 35-54.
110. Mouritz, A.P. and Gibson, A.G. (2006) *Fire Properties of Polymer Composite Materials*, Springer Ltd, Dordrecht, the Netherlands.
111. Naus, D.J. (2006) "The effect of elevated temperatures on concrete materials and structures - A literature review." U.S. Nuclear Regulatory Commission, Office of Nuclear Regulatory Research, Washington. D.C. 20555-0001.
112. Neves, I.C., Rodrigues, J.C. and Loureiro, A.P. (1996) "Mechanical properties of reinforcing and prestressing steels after heating." *Journal of Materials in Civil Engineering*, 8(4), 189-194.
113. Nomura, S. and Ball, D.L. (1993) "Micromechanical formulation of effective thermal expansion coefficients of unidirectional fiber composites." *Advanced Composite Materials*, 3(2), 143-152.
114. Oehlers, D.J., Haskett, M., Wu, C., and Seracino, R. (2008) "Embedding NSM FRP plates for improved IC debonding resistance." *Journal of Composites for Construction*, 12(6), 635-642.
115. Palmieri, A., Matthys, S. and Taerwe, L. (2011) "Influence of high temperature on bond between NSM FRP bars/strips and concrete." *Proceedings of the 10th International Symposium of the Fiber-Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, Tampa, FL, 919-930.

116. Palmieri, A., Matthys, S. and Taerwe, L. (2012) "Experimental investigation on fire endurance of insulated concrete beams strengthened with near surface mounted FRP bar reinforcement." *Composite Part B*, 43(3), 885-895.
117. Phan, L.T. (1996) "Fire performance of high-strength concrete: A report of the state-of-the-art." NISTIR 5934, National Institute of standard and Technology, Gaithersburg, MD.
118. Phan, L.T., Lawson, J.R. and Davis, F.L. (2000) "Heating, spalling characteristics and residual properties of high performance concrete." *15th Meeting of UJNR Panel on Fire Research and Safety*, Vol. 2, NIST, USA.
119. Pirgon, O., Wostenholm, G.H. and Yates, B. (1973) "Thermal expansion at elevated temperatures – IV Carbon-fiber composite." *Journal of Physics D: Applied Physics*, 6(1), 309-321.
120. Priestley, M.J.N., Seible, F., Xiao, Y., and Verma, R. (1994) Steel jacket retrofit of squat RC bridge columns for enhanced shear strength – Part I – Theoretical considerations and test design. *ACI Structural Journal*, 91(4), 394-405.
121. Rafi, M.M., Nadjai, A. and Ali, F. (2007) "Fire resistance of carbon FRP reinforced concrete beams." *Magazine of Concrete Research*, 59(4), 245-255.
122. Rafi, M.M., Nadjai, A. and Ali, F. (2008) "Finite element modeling of carbon fiber-reinforced polymer reinforced concrete beams under elevated temperatures." *ACI Structural Journal*, 105(6), 701-710.
123. Raghavan, J. and Meshii, M. (1997) "Creep of polymer composites." *Composites Science and Technology*, 57(12), 1673-1688.
124. Raman, A.H., Taylor, D.A., and Kingsley, C.Y. (1993) "Evaluation of FRP as reinforcement for concrete bridges." *Symposium on Fiber-reinforced plastic reinforcement for concrete structures (FRPRCS)*, American Concrete Institute, Detroit, MI, 71-86.
125. Rasheed, H.A., Harrison, R.R., Peterman, R.J. and Alkhrdaji, T. (2010) "Ductile strengthening using externally bonded and near surface mounted composite systems." *Composite Structures*, 92(10), 2379-2390.
126. Rashid, R., Oehlers, D.J. and Seracino, R. (2008) "IC debonding of FRP NSM and EB retrofitted concrete: plate and cover interaction tests." *Journal of Composites for Construction*, 12(2), 160-167.
127. Raut, N. and Kodur, V.K.R. (2011) "Response of Reinforced Concrete Columns under Fire-Induced Biaxial Bending." *ACI Structural Journal*, 108(5), 610-619.

128. Rein, G., Abecassis, E.C. and Carvel, R. (2007) "The Dalmarnock Fire Tests: Experiments and Modeling." School of Engineering and Electronics, University of Edinburgh.
129. Saafi, M. (2002) "Effect of fire on FRP reinforced concrete members." *Composite Structures*, 58(1), 11–20.
130. Sakashita, M., Masuda, Y., Nakamura, K. and Tanano, H. (1997) "Deflection of continuous fiber reinforced concrete beams subject to loaded heating." *Proceedings of 3th International Symposium on Non-Metallic (FRP) Reinforcement for Concrete Structures*, Vol. 2, Sapporo, Japan, 51-58.
131. Sauder, C., Lamon, J. and Pailier, R. (2004) The Tensile Behavior of Carbon Fibers at High Temperatures up to 2400°C, *Carbon*, 42(4), 715-725.
132. Savva, A., Manita, P. and Sideris, K.K. (2005) "Influence of elevated temperature on the mechanical properties of blended cement concretes prepared with limestone and siliceous aggregates." *Cement and Concrete Composites*, 27(2), 239-248.
133. Schapery, R.A. (1968) "Thermal expansion coefficients of composite materials." *Journal of Composite Materials*, 2(3), 380-404.
134. Schneider, U. (1988) "Concrete at high temperatures – A general Review." *Fire Safety Journal*, 13(1), 55-68.
135. Sena Cruz, J.M. and Barros, J.A.O. (2002) "Bond behavior of carbon laminate strips into concrete by pull-out bending tests." *Proceedings of the International Symposium - Bond in Concrete - from Research to Standards*, Budapest, Hungary, 614-621.
136. Sena-Cruz, J.M. and Barros, J.A.O. (2004) "Bond between near-surface mounted carbon-fiber- reinforced polymer laminate strips and concrete." *Journal of Composites for Construction*, 8(6), 519-527.
137. Shokralla, S. A. and Al-Muaiikel, N. S. (2010) "Thermal properties of epoxy (DGEBA)/phenolic resin (Novolac) blends." *Arabian Journal for Science and Engineering*, 35(1), 7-14.
138. Society of Fire Protection Engineers (2008) "SFPE Handbook of Fire Protection Engineering." National Fire Protection Agency, Quincy, MA.
139. Swamy, R.N., Jones, R. and Bloxham, J.W. (1987) "Structural behavior of reinforced concrete beams strengthened by epoxy-bonded steel plates." *Structural Engineer*, 65(2), 59-68.

140. TA Instruments (2011) Help document - Thermal Mechanical Analyzer. New Castle, DE.
141. Taljsten B., Carolin A. and Nordin H. (2003) "Concrete structures strengthened with near surface mounted reinforcement of CFRP." *Advances in Structural Engineering*, 6(3), 201-213.
142. Teng, J.G., Chen, J.F., Smith, S.T. and Lam, L. (2002) FRP-strengthened RC structures, John Wiley & Sons Inc. Chichester, UK.
143. Teng, J.G., De Lorenzis, L., Wang, B., Rong, L., Wong, T.N. and Lam, L. (2006) "Debonding failures of RC beams strengthened with near-surface mounted CFRP strips." *Journal of Composite for Constructions, ASCE*, 10(2), 92-105.
144. Teng, J.G., Smith, S.T., Yao, J., and Chen, J.F. (2003) "Intermediate crack-induced debonding in RC beams and slabs." *Construction and Building Materials*, 17(6), 447-462.
145. Thomas, F.G. and Webster, C.T. (1953) "Fire resistance of reinforced concrete columns." *National Building Studies Research Paper No 18*, HMSO, London, UK.
146. Wang, Y.C., Wong, P.M.H., and Kodur, V. (2007) "An experimental study of the mechanical properties of fibre reinforced polymer (FRP) and steel reinforcing bars at elevated temperatures." *Composite Structures*, 80(1), 131-140.
147. Wickstrom, U. (1986) "A very simple method for estimating temperatures." *Fire Exposed Structures, in New Technology to Reduced Fire Losses and Costs*. Elsevier Applied Science, London, UK, 186-194.
148. William, B. and Richard, D. (1990) A first course in the finite element method, Irwin, Inc., Boston, MA.
149. Williams, B. K. (2004). "Fire performance of FRP-strengthened reinforced concrete flexural members." Doctoral Thesis, Queen's University, Kingston, Canada.
150. Williams, B. Kodur, V., Green, M.F. and Bisby, L.A. (2008) "Fire endurance of fiber-reinforced polymer strengthened concrete T-beams." *ACI Structural Journal*, 105(1), 60-67.
151. Wu, Z. S., Iwashita, K., Yagashiro, S., Ishikawa, T., and Hamaguchi, Y. (2004). "Temperature effect on bonding and debonding behaviour between FRP sheets and concrete." *FRP Composites in Civil Engineering (CICE)*, 905-912.

Yu, B. and Kodur, V.K.R. (2013) “Factors governing the fire response of concrete beams reinforced with FRP rebars.” *Composite Structures*, Vol.100, 257-269.