Mis-conceptions regarding Use of GFRP as Internal Reinforcement

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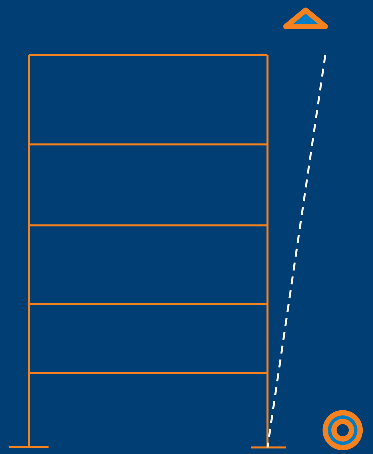
5 Major Mis-conceptions Regarding GFRP

- Lack of ductility since GFRP is linear elastic until fracture.
- Lack of fire resilience since GFRP is like plastic.
- Brittle under cold temperature.
- Need 3 times as much of GFRP compared with steel reinforcement due to low modulus of 60 GPa versus 200 GPa for steel.
- It is a hidden time bomb due to long term degradation in alkaline and susceptibility to creep rupture.

1. What is Structural Ductility?

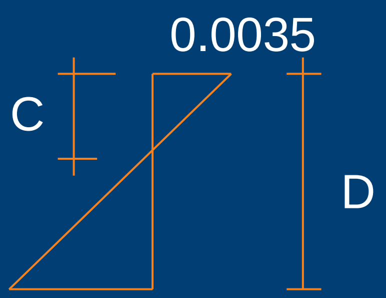
- The ability of a structural system or element to undergo large deformation and sustain service load without failure
- Structural system ductility vs sectional ductility



Structural displacement

Sectional rotation

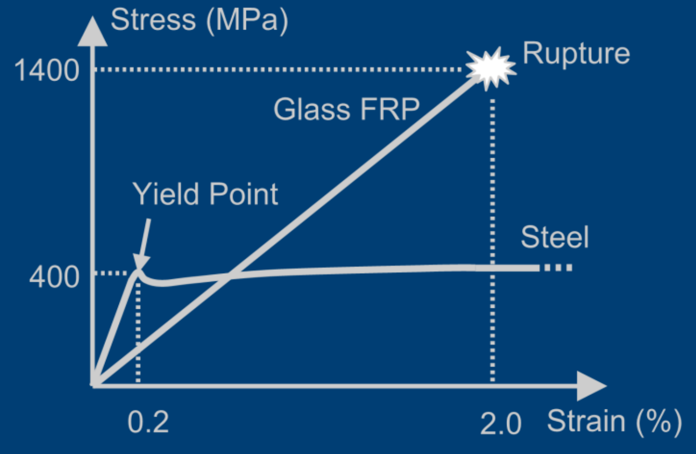
The required sectional ductility could be several times that of the overall structural displacement ductility.
This is a typical concern for seismic design.



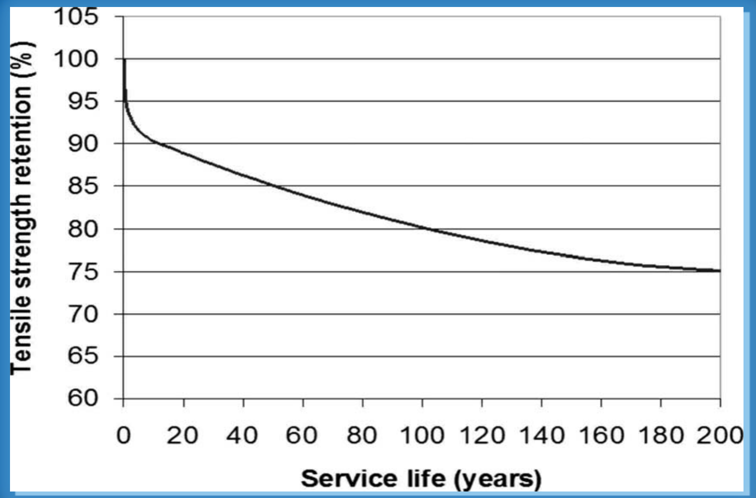
C/D determines sectional ductility

Key Differences between GFRP and Steel Bars

- GFRP is linear elastic up to rupture, steel is linear elastic up to yield, and plastic up to ultimate failure.
- GFRP strength changes over time. While steel corrodes if exposed to chlorides, its material properties do not change. Lower Φ factor for GFRP design accounts for strength degradation due to environmental exposure.



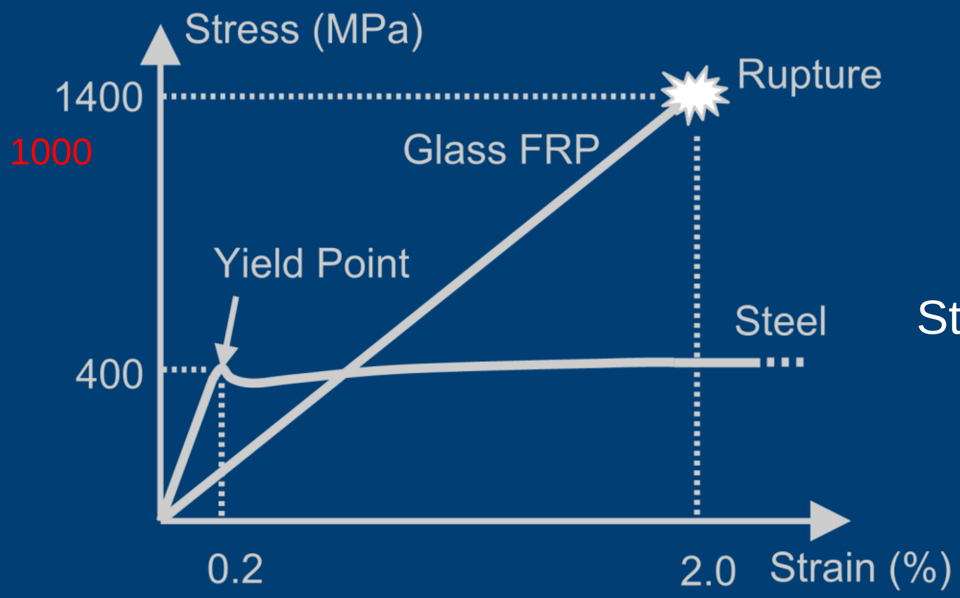
Stress strain curves for GFRP and steel reinforcement (ISIS 2006)



General relation between tensile strength retention and predicted service life at mean annual temperature of 6˚ C Montreal (Robert et al. 2009)

Deformability (§ 16.8.2.1) CHBDC

- Deformability takes into account absorbed energy based on deformability, to ensure members reinforced with FRP can sustain adequate deformation prior to failure.
- It is desirable that a FRP reinforced section would have comparable deformability as expected of a steel reinforced section. However, since GFRP does not yield, deformability is expressed in terms of the total strain energy at failure over the strain energy at service load.



Stress strain curves for GFRP and steel reinforcement (ISIS 2006)

Deformability (§ 16.8.2.1) CHBDC

- Overall performance factor, J, must be at least 4.0 for rectangular sections and 6.0 for T sections.

$$J = \frac{M_{ult}\psi_{ult}}{M_c\psi_c}$$

- M_{ult} and Ψ_{ult} are moment and curvature at ultimate limit state.

M_{ult} = moment at ultimate limit state

$\Psi_{ult} = \varepsilon_{ult} / kd$

- M_c and Ψ_c are moment and curvature corresponding to a concrete strain of 0.001.

$M_c = f_c\, k\, (1-k/3)\, bd^2$

$\Psi_c = \varepsilon_c / kd$, where $\varepsilon_c = 0.001$

- Use $f_c = \varepsilon_c E_c$ or $f_c = 1.8 f'_c (\varepsilon_c/\varepsilon'_c) / (1+(\varepsilon_c/\varepsilon'_c)^2)$

Deformability (§ 16.8.2.1)

- For calculating M_{ult} and Ψ_{ult}, repeat the same steps as required to calculate M_r, but with higher resistance factors.
- For calculating M_{ult} and Ψ_{ult} use the following for MTO bridges:

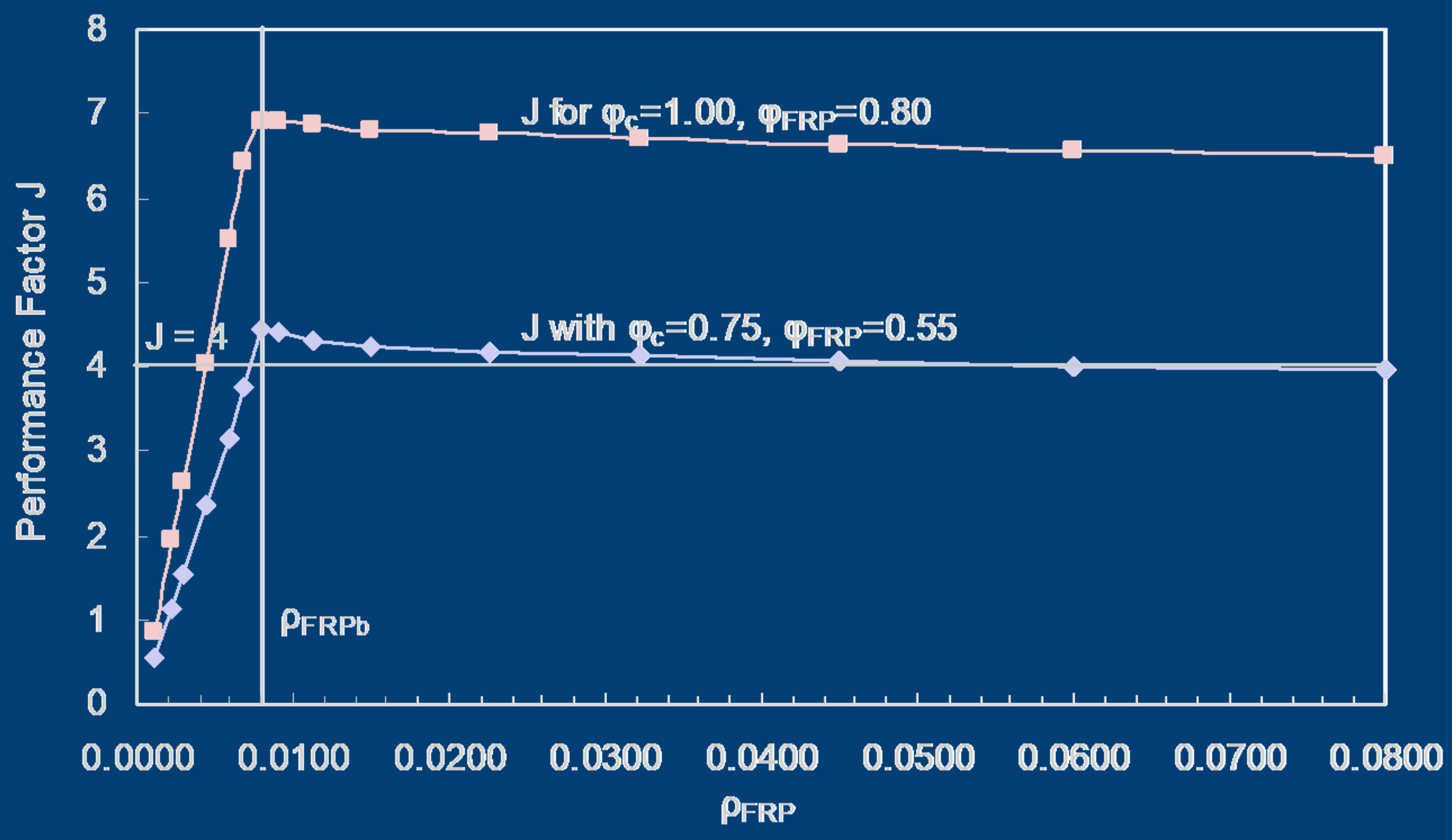
$\Phi_c = 1.00$

$\Phi_{FRP} = 0.80$

- Based on the definition of J as a function of M_c, tension-controlled members may not have adequate deformability.
- Deformability may govern the design of deep members or T-beam members (i.e. pier caps or diaphragms).

Performance Factor

- Performance factor J of a 500 mm thick slab reinforced with varying quantity of G1-15 bars.

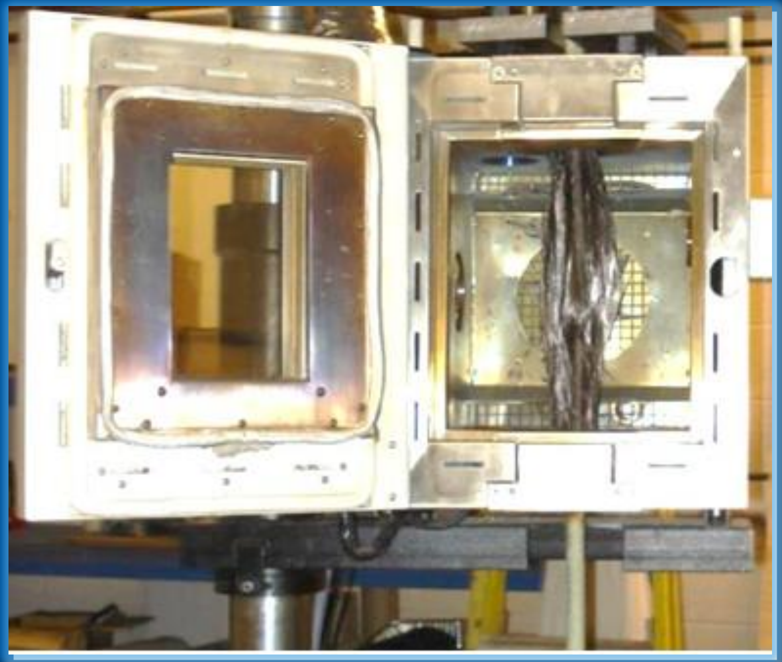


2. Fire Resilience

- Tensile capacity under elevated temperature
- Bond capacity under elevated temperature
- Residual capacity after fire

Tensile Tests

Queens University, Ontario, Canada, Professor Mark Green

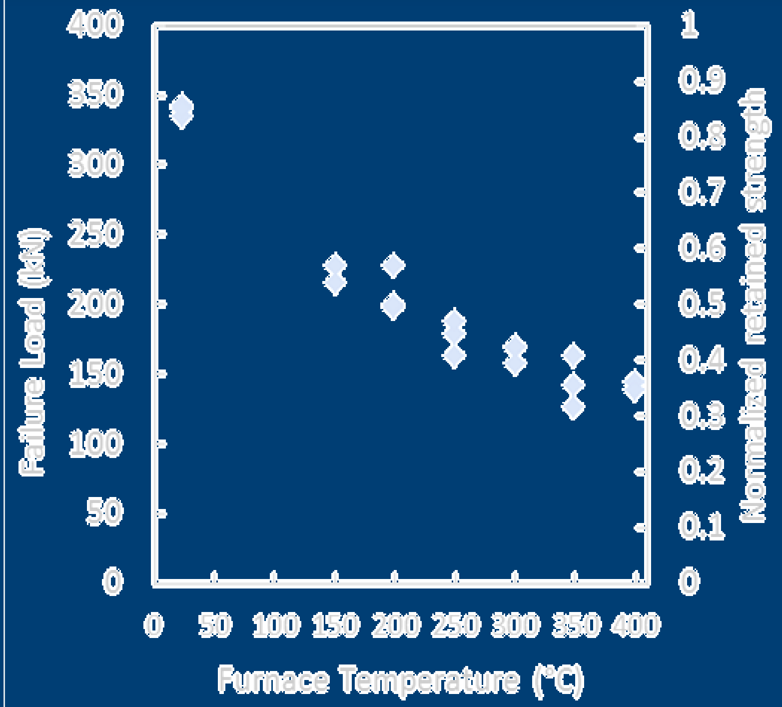


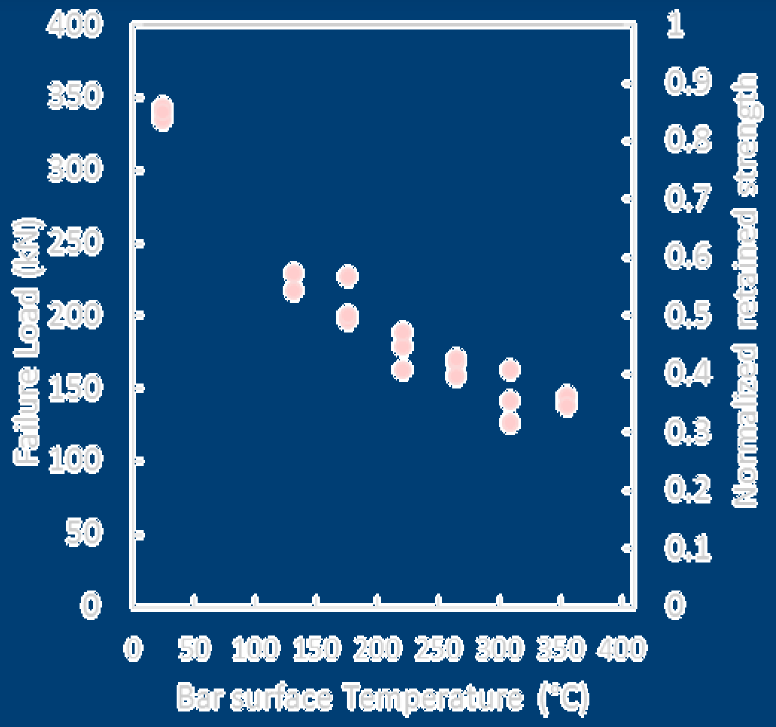
Test setup: (Left) Fixture; (Right) Heating chamber

Tensile Tests (cont.)

Queens University, Ontario, Canada, Professor Mark Green

Steady state
Product A



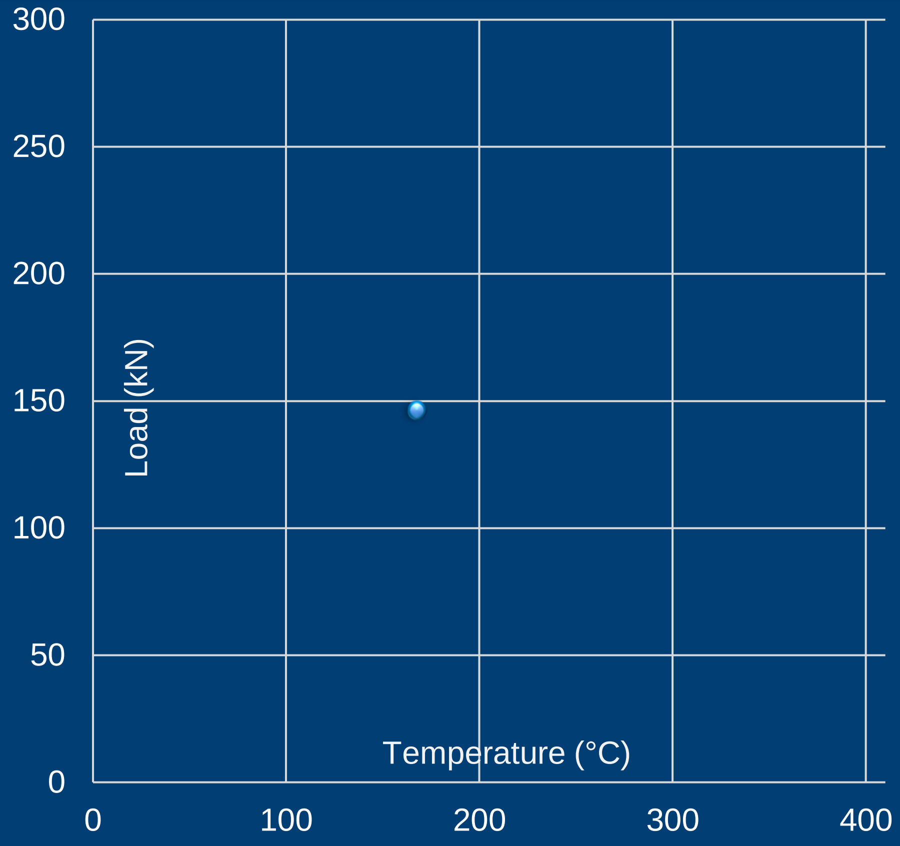


Tensile strength degradation in steady state tests (#5 GFRP bars):
(Left) as a function of furnace temperature;
(Right) as a function of bar surface temperature

Tensile Tests (cont.)

Queens University, Ontario, Canada, Professor Mark Green

Steady state
Product B



Tensile Tests (cont.)

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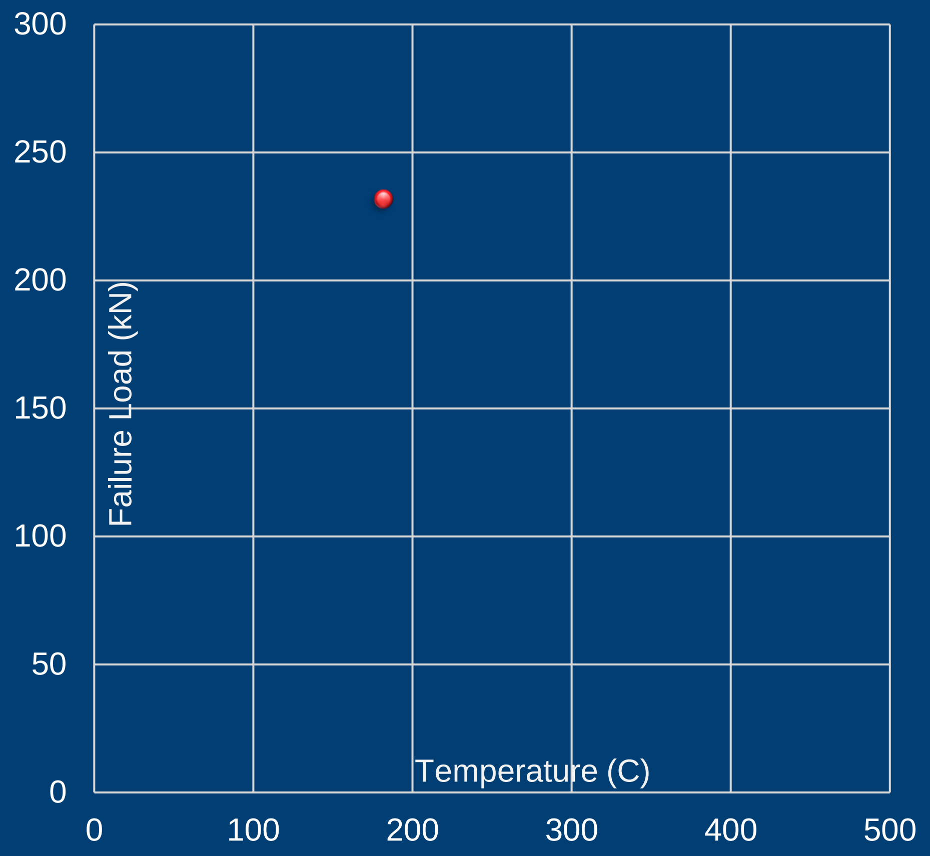
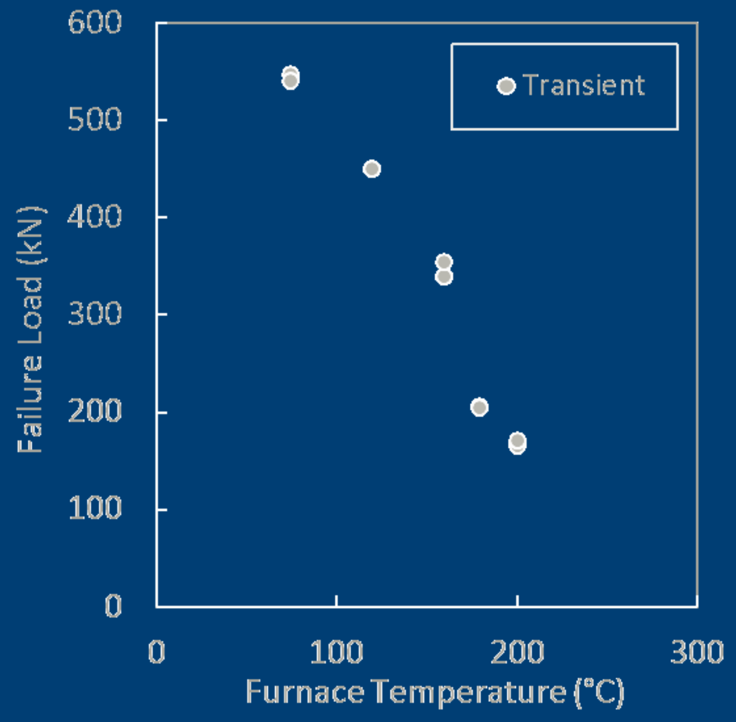
Steady state
Product C

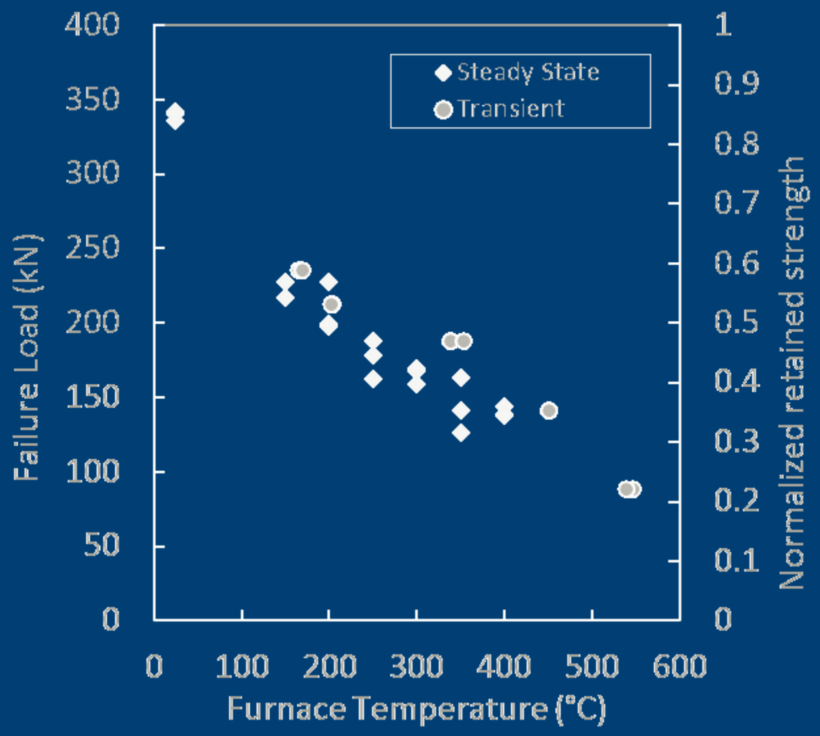


Tensile Tests (cont.)

Queens University, Ontario, Canada, Professor Mark Green

Transient condition tests
Product A





(Left) Tensile strength degradation in Transient tests (#5 GFRP bars);
(Right) comparison of transient test results to steady state tests

Pull-Out Tests

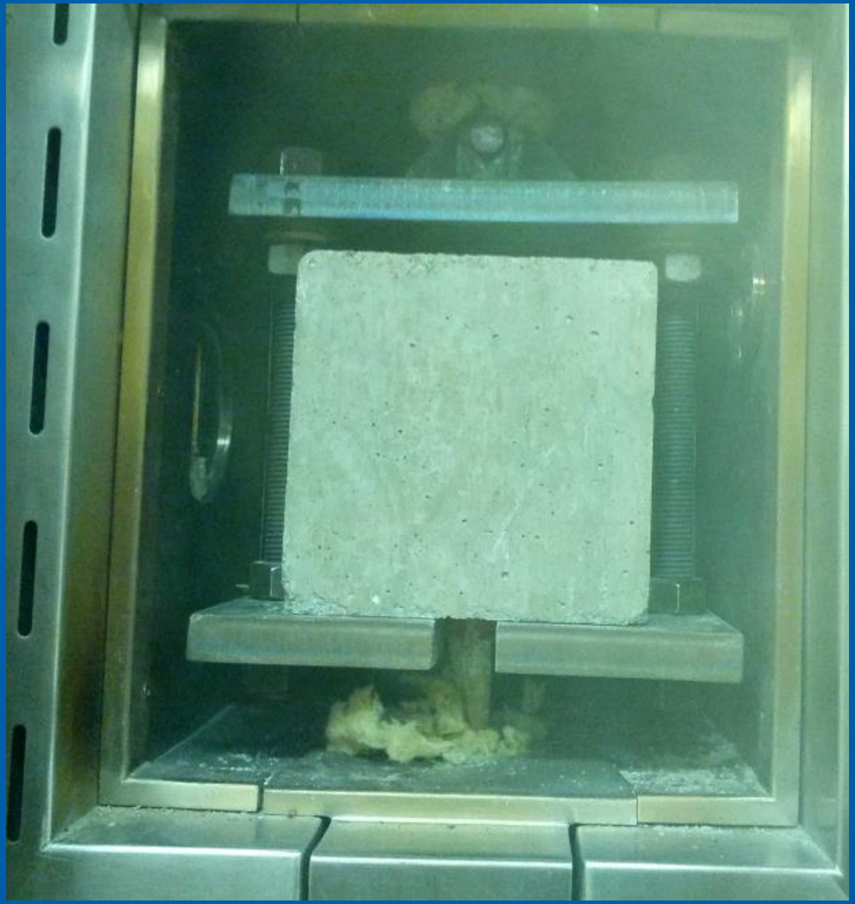
Queens University, Ontario, Canada, Professor Mark Green

Sample Fabrication

Thermocouple placements (Right); bond breaker (Middle); concrete casting (Left)

Pull-Out Tests (cont.)

Queens University, Ontario, Canada, Professor Mark Green



Pullout sample placed in furnace

Pull-Out Tests (cont.)

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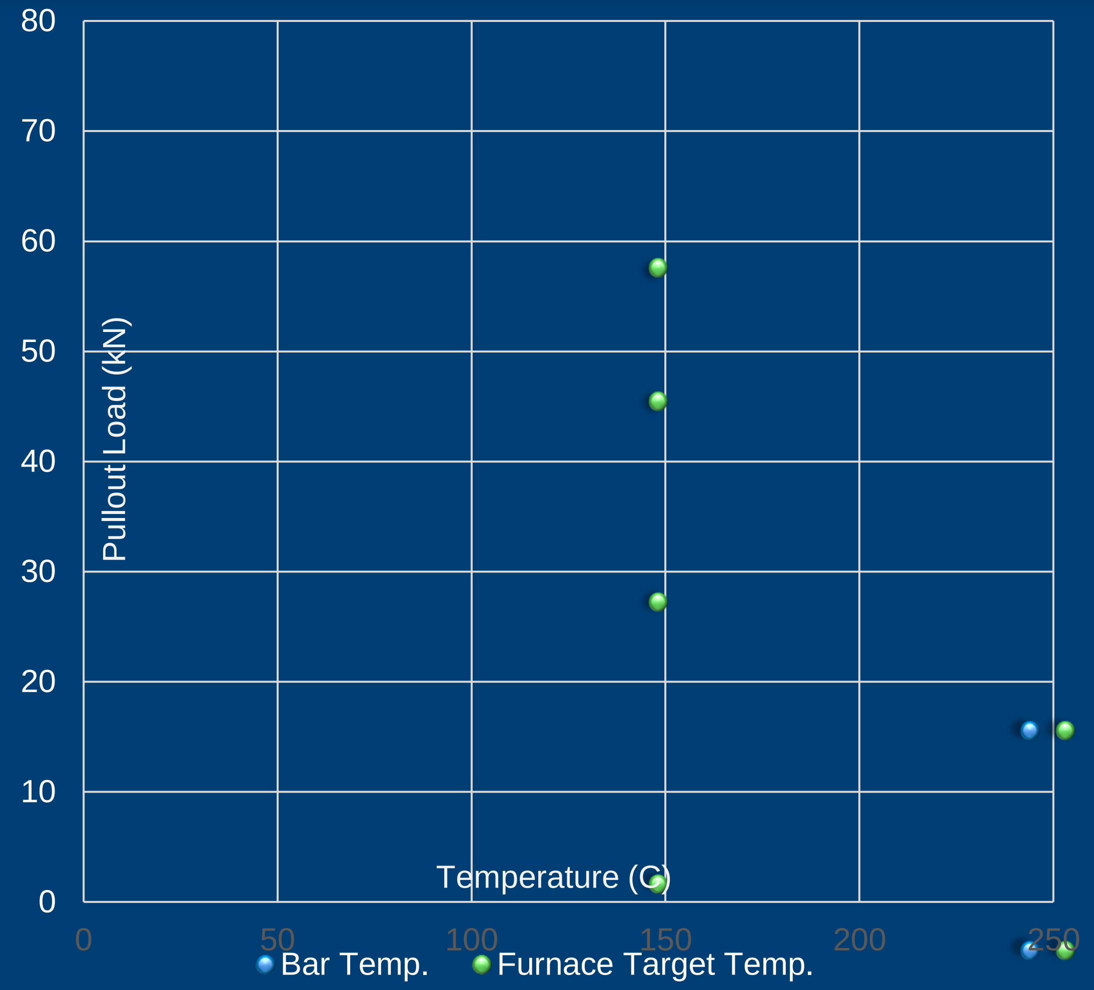
Steady state
Product A



Pull-Out Tests (cont.)

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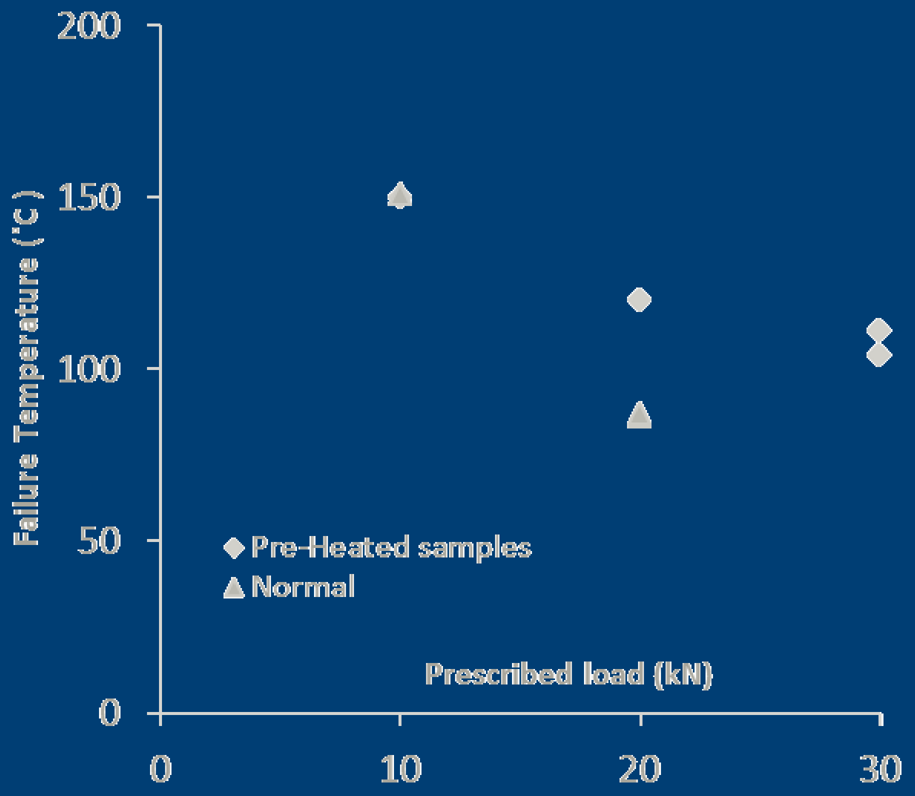
Product A
Transient condition



Transient condition pull-out test results

Pull-Out Tests (cont.)

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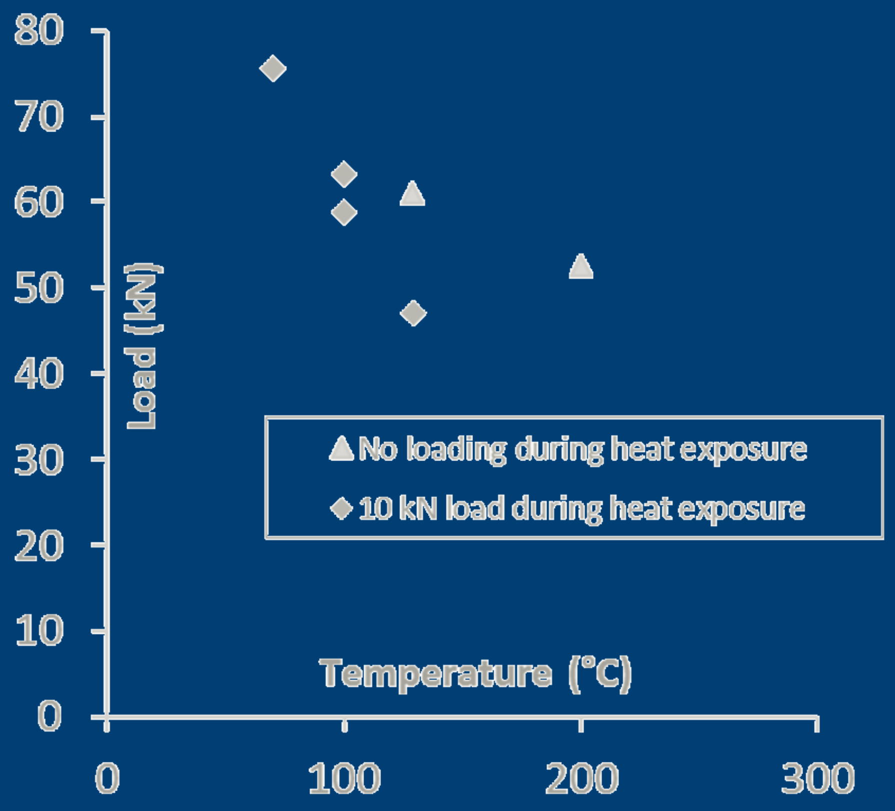
Product A
Residual tests



Residual strength (after fire) pullout test results

GFRP Reinforced Concrete Bridge Decks in Fire Scenarios

Hamzeh Hajiloo and Mark Green
Civil Engineering, Queen's University, 58 University Avenue, Kingston ON Canada K7L 3N6

(2018b_ENREF_19).

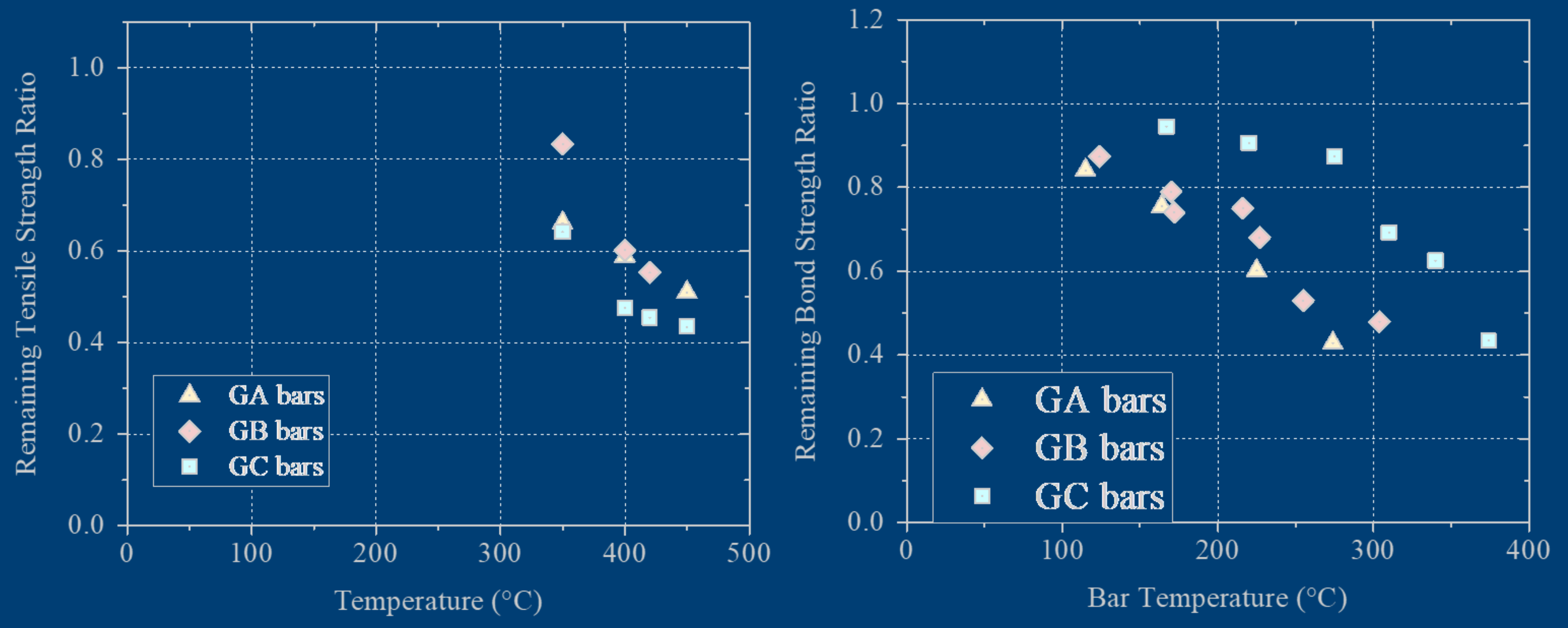


Fig. 1 Remaining residual properties of GFRP bars: (a) tensile strength; (b) bond strength

Fire Test on Slab with 60mm cover

ACMBS MCAPC VII	**7th International Conference on Advanced Composite Materials in Bridges and Structures** **7ième Conférence Internationale sur les matériaux composites d'avant-garde pour ponts et charpentes** **Vancouver, British Columbia, Canada, 24 – 26 August 2016 / 24 – 26 août 2016**

FIRE PERFORMANCE OF FRP REINFORCED CONCRETE SLABS

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Noureddine Bénichou and Mohamed Sultan
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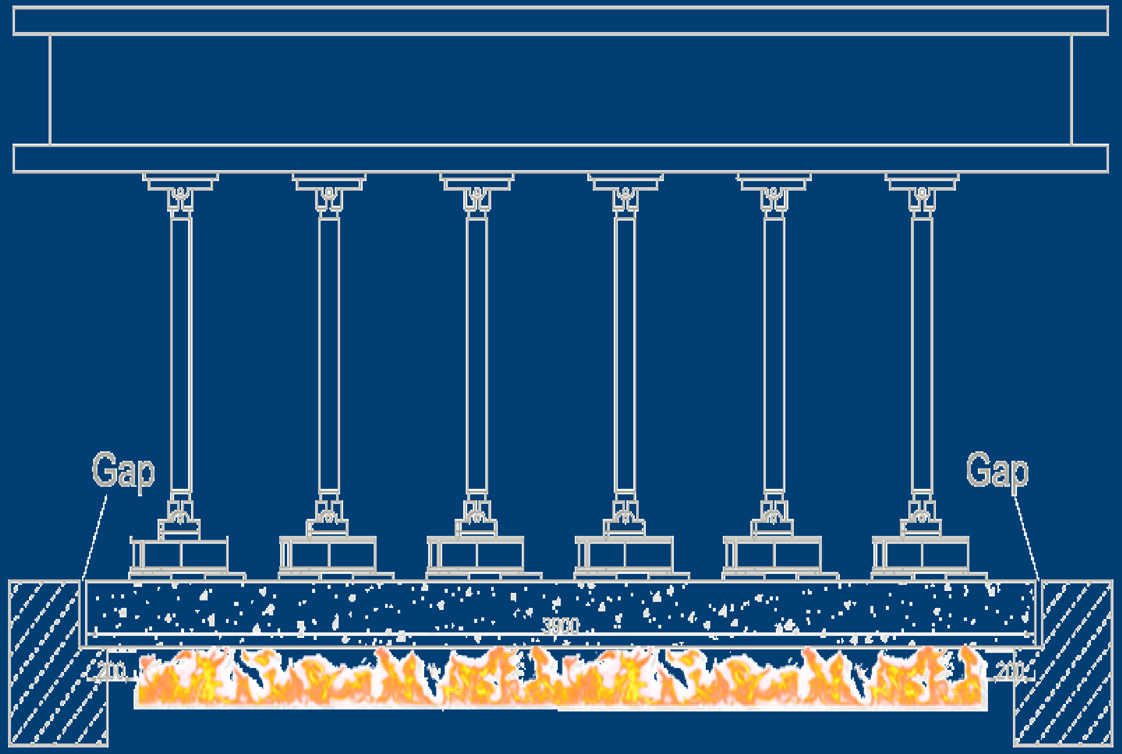
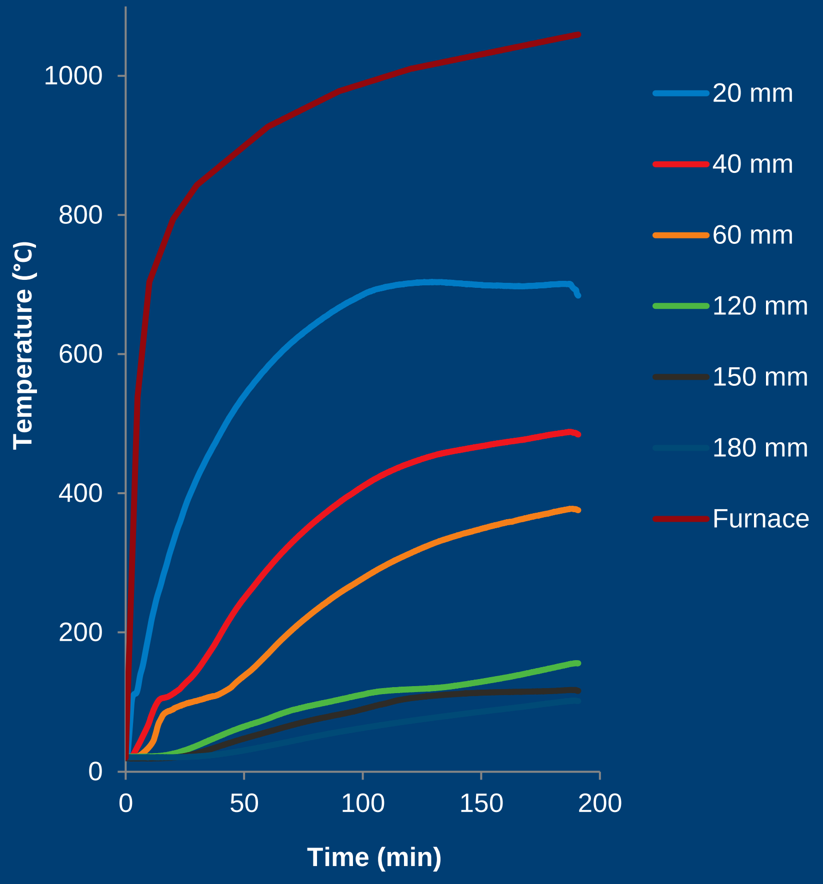
Fire Tests (cont.)



Test Setup at NRC– slab 1200 x 3900 x 200mm thick
#5@100 Bot with 60mm cover



Fire test result for slab with 60mm cover

- Test slabs were loaded to 53% of ultimate capacity.
- Both slabs were able to carry the superimposed load for more than 180 minutes. At 184 minutes, the load was increased from 19.2 kN/m to 23.1 kN/m causing Slab-A to fail.
- It should be noted that the load applied on the slabs during fire exposure was well beyond the expected load on the slabs in a real fire incident.

Fire Test on Slab with 40mm Cover

Hamzeh Hajiloo[1], Mark F. Green[1], Noureddine Bénichou[2], Mohamed Sultan[2]
[1] Civil Engineering, Queen's University, 58 University Avenue, Kingston ON Canada
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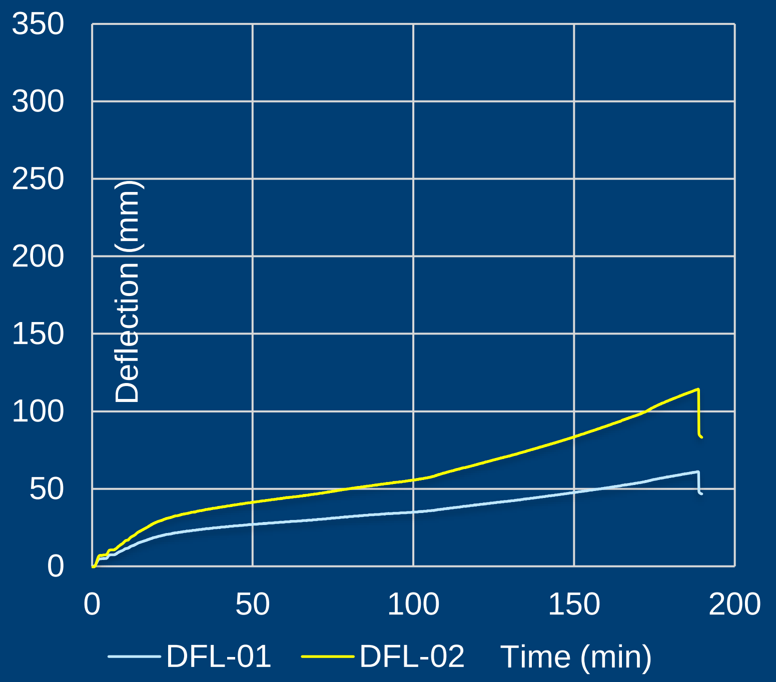
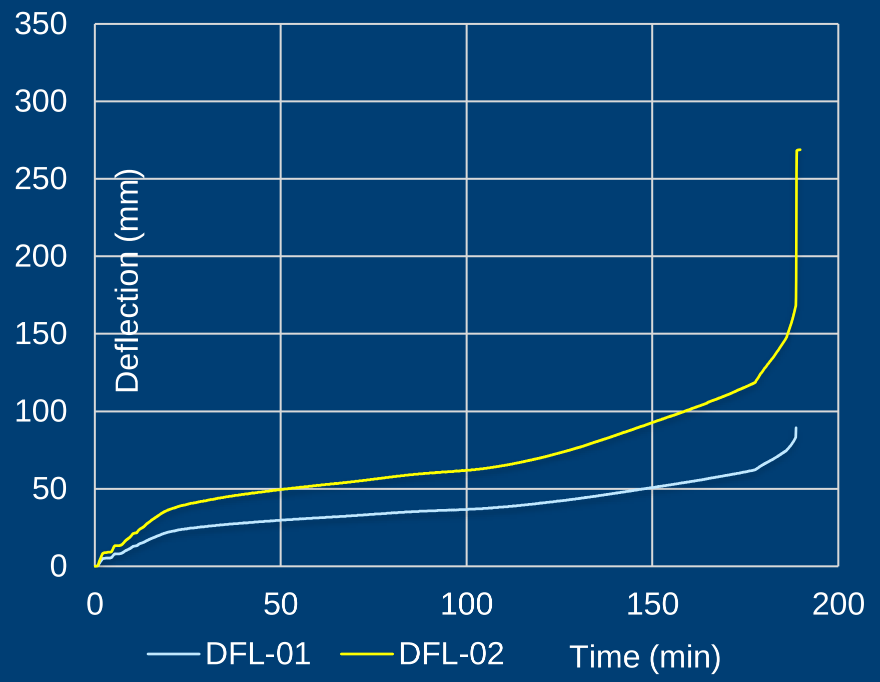




Both slabs sustained load equal to 45% of ultimate capacity for more than 180 minutes.

Lessons learned from fire test of GFRP / slabs

- Bond degradation is the controlling factor; at 170 degree C the remaining bond strength is only 10%
- At higher temperature, the GFRP will act like unbonded external reinforcement with a catenary profile
- As long as there is an end anchorage zone around 200mm that is shielded from the heat, a one-way slab with 40mm of cover to the GFRP can sustain the service load up to 3 hours.
- The slab will suffer large permanent deformation and therefore likely not serviceable after the fire.

Fire resistance of FRP-RC structures according to ACI 440 draft

(B) Fire Protection in One-Way Slab

7.2.4 The effects of fire shall be considered in design.

R7.2.4 *The performance of GFRP- reinforced concrete elements at high temperatures relies primarily on the GFRP-concrete bond strength being maintained (Hajiloo and Green 2018, Hajiloo et al. 2019, Hajiloo et al. 2017, Nigro et al. 2011). Table R20.6.1.3.1 provides the fire-resistance ratings for the concrete covers specified in Table 20.6.1.3.1 for nonbond-critical GFRP reinforcement.*

(B-1) Detailing to obtain nonbond-critical GFRP reinforcement

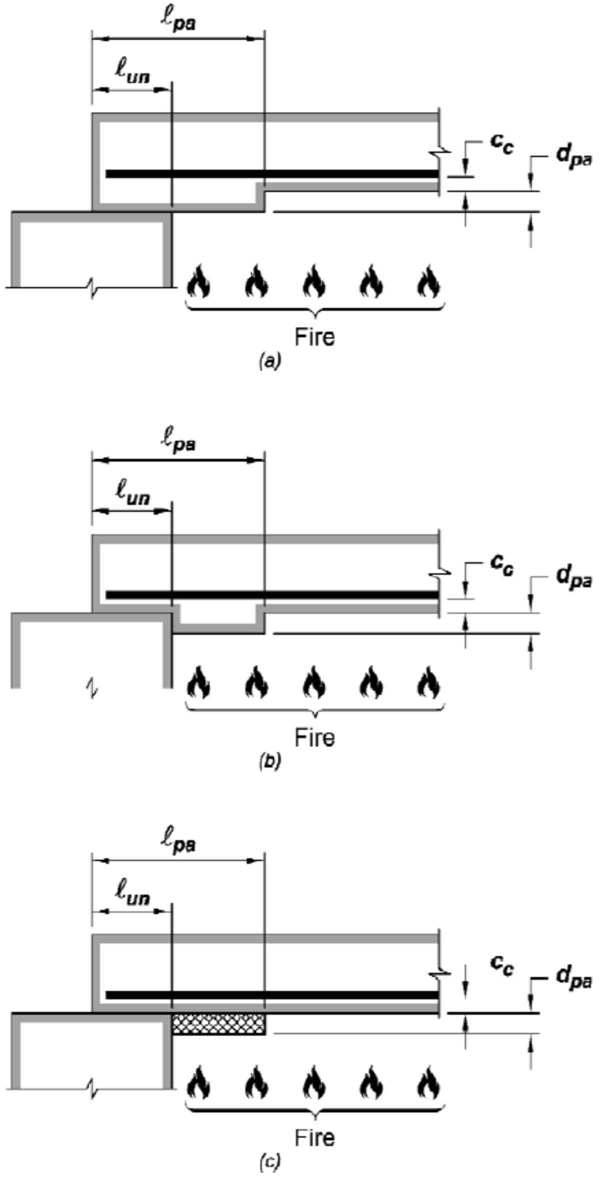


Fig R7.2.4a-c—Protection of GFRP reinforcement near supports.

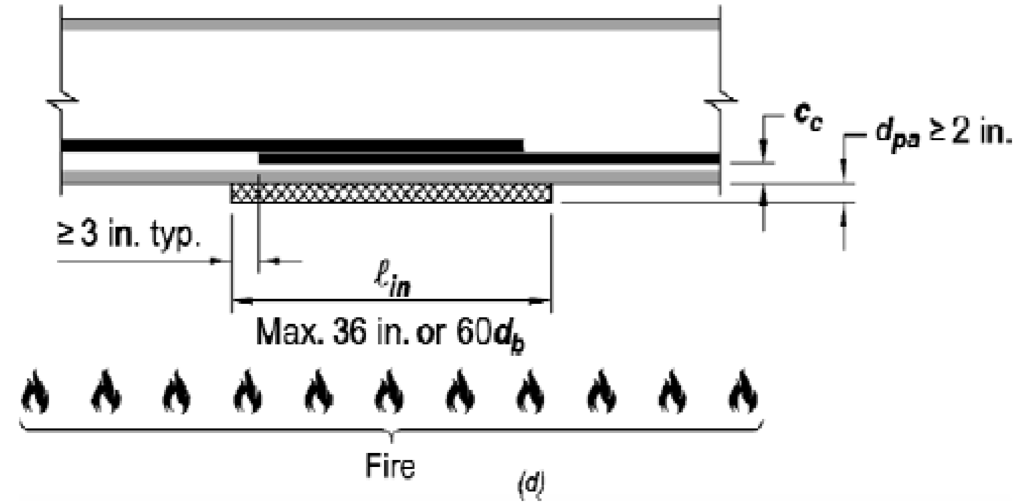


Fig R7.2.4.d—Insulation at spliced GFRP reinforcement.

3. Brittleness under Cold Temperature

- Steel suffers significant reduction in ability to absorb impact energy under cold temperature, leading to charpy V-notch energy requirements for cold temperature applications
- How would GFRP behave?
- MTO conducted Charpy V-notch impact tests on GFRP in 2006 on two products, #5, #6 and #8 GFRP rebars

Fig. 1: CVN specimen before test

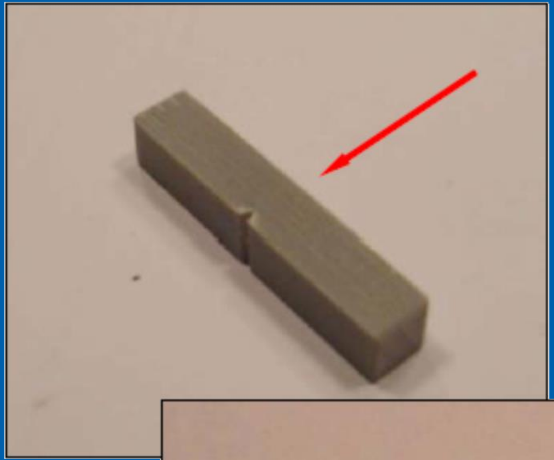
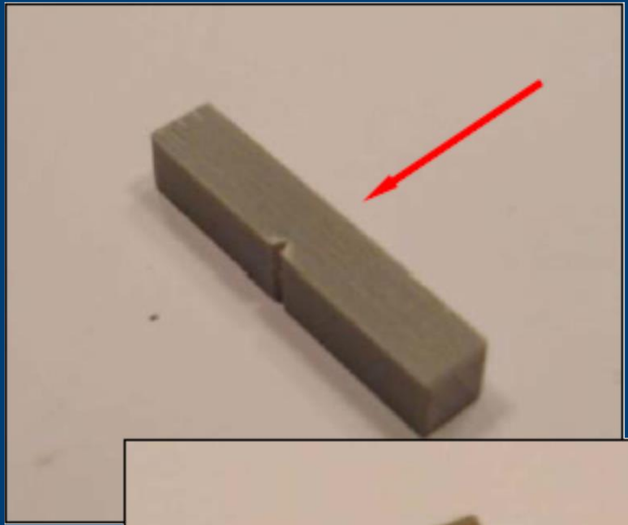


Fig. 2: Fractured CVN specimen after the test

Impact Test Results (ASTM E23) *Table 1*

Test Temperature	Sample ID	Test Results (ft·lbf)		
		Ø0.625 Rod	Ø0.750 Rod	Ø1.000 Rod
26° C	1	30	36	36
	2	33	29	28
	3	28	37	23
	Average	***30***	***34***	***29***
-20°C	1	38	41	36
	2	38	40	39
	3	33	38	36
	Average	***36***	***40***	***37***
-50°C	1	38	45	43
	2	37	41	41
	3	39	41	33
	Average	***38***	***42***	***39***

Fig. 1: CVN specimen before test



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	3	26	28	29
	Average	*26*	*27*	*28*
-20°C	1	30	31	27
	2	34	24	28
	3	32	30	30
	Average	*32*	*28*	*28*
-50°C	1	29	35	30
	2	36	29	30
	3	37	33	33
	Average	*34*	*32*	*31*

4. Amount of GFRP Vs Steel Reinforcement

- It is true that GFRP design is usually governed by SLS
- Do we really need 3X of steel area in GFRP because of modulus ratio?
- Need to calculate crack width and deflection to compare with steel

$$w_{cr} = 2\frac{f_{FRP}}{E_{FRP}}\frac{h_2}{h_1}k_b\sqrt{d_c^2 + (s/2)^2}$$

Section 16
CHBDC

d_c = distance from the centroid of the tension reinforcement to the extreme tension surface of concrete, mm

h_1 = distance from the centroid of tension reinforcement to the neutral axis, mm

h_2 = distance from the extreme flexural tension surface to the neutral axis, mm

s = spacing of shear or tensile reinforcement, mm

Example: One-way slab

Comparing Grade 400W steel reinforcement Vs High Modulus GFRP

- Span = 4000mm
- Thickness = 200mm
- DL = 4.8 KPa

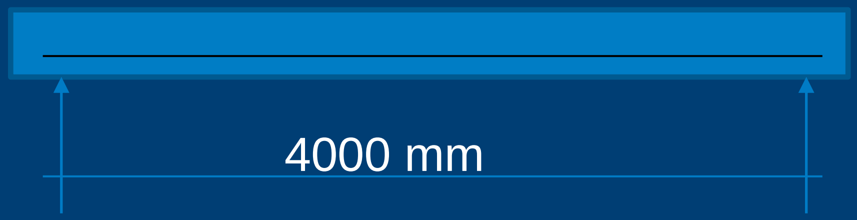
  SDL = 0.6 KPa

  LL = 4.8 Kpa
- Mf = 31.4 KN.m/m ULS

  Ma = 20.4 KN.m/m SLS

  Mdl = 10.8 KN.m/m

  Mcr = 14.7 KN.m/m



Load effects	15M @ 300 steel	15M @300 GFRP	13M @ 150 GFRP
Icr	10.6 x 10^6 mm^4	12.4 x 10^6 mm^4	16.4 x 10^6 mm^4
Tensile stress rebar	221 MPa	213 MPa	163 MPa
Crack width	0.18 mm	1.2 mm	0.5 mm
Mr at ULS	34.5 KN.m/m	61 KN.m/m	
LL Deflection	2.1 mm	8.7 mm	6.7 mm

- Deflections for GFRP according to ACI 440 modified Branson's Equation for I effective
- A one-to-one substitution from steel to GFRP would not be adequate for crack control and likely not adequate for deflection control.
- An increase of around 30% of reinforcing area with half the spacing would solve the problem for this slab.
- Since GFRP does not corrode, the 0.5mm crack width limit is mainly for aesthetics.

5. Is it a “Time Bomb”?

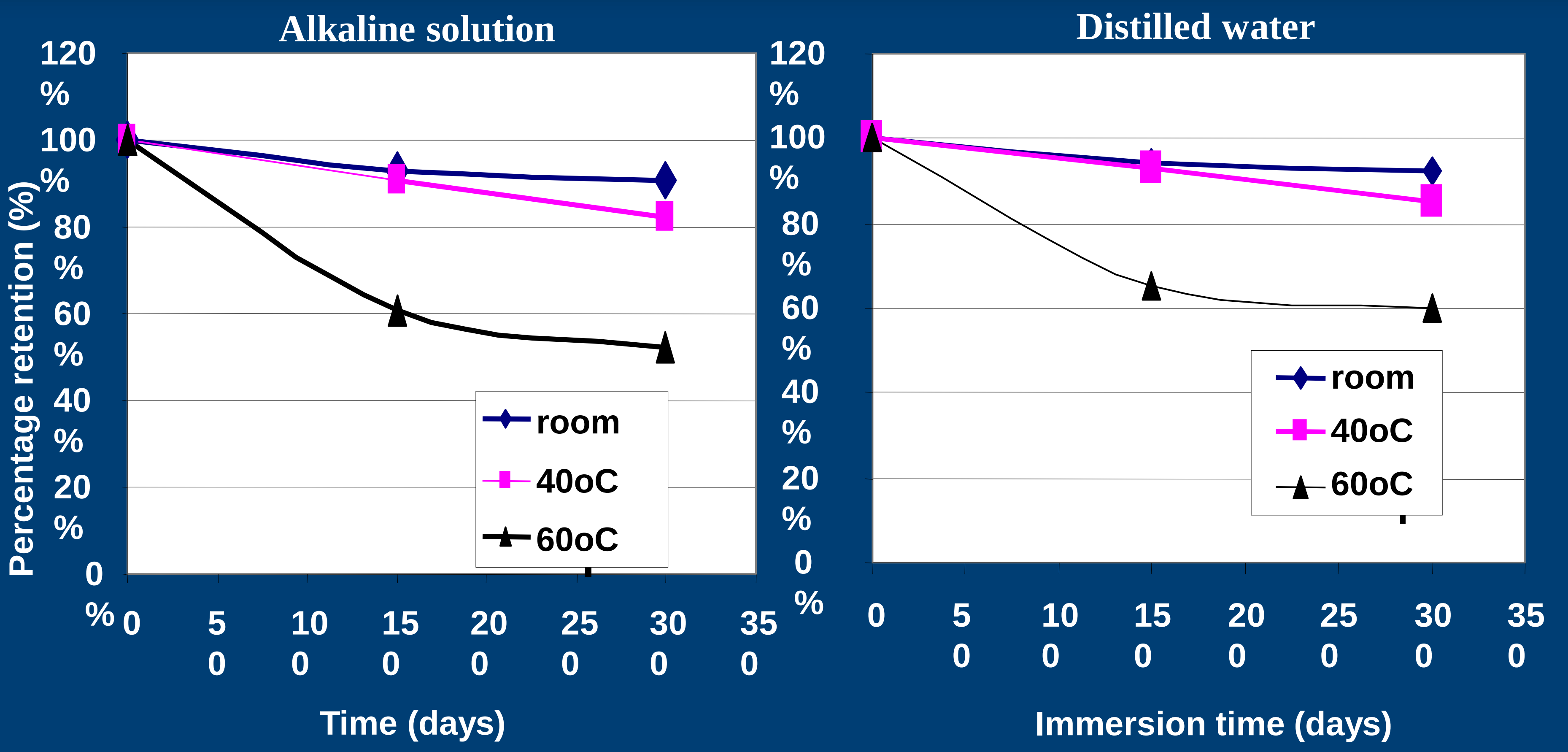
- How much degradation is expected over a 75 year design life?
- Creep and fatigue?
- How does it compare with epoxy coated rebars?

Accelerated aging and natural aging condition

$$\frac{N}{C} = 0.098\, e^{0.0558T}$$

Temperature (ºC)	Solution (pH 12.6-12.8)	Accelerated ages (days)	Natural ages (years)
40	Alkaline	150	13
40	Alkaline	300	27
60	Alkaline	150	100
60	Alkaline	300	199

Typical Results of 1st Generation Research (15.9mm diameter)



Two competing mechanisms: post-cure and degradation.

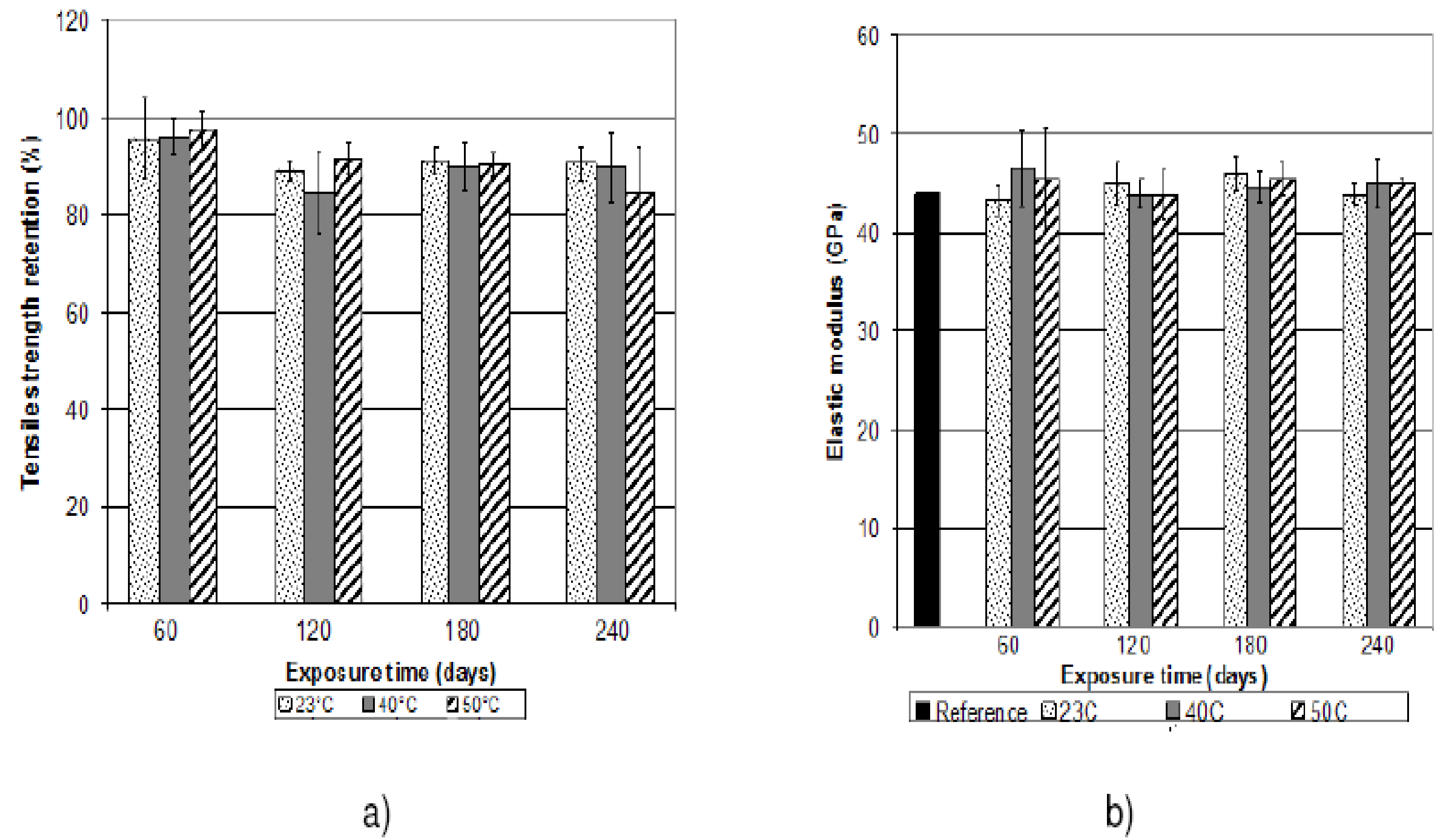


Figure 3 Tensile properties retention of conditioned GFRP bars aged in moist concrete at 23°, 40°, and 50°C: a) tensile strength; b) elastic modulus.

Robert and Benmokrane FRPRCS-9 (2009)

Recent test results of high-performance product

Specimen	Lot #	Ultimate Load (kN)	Ultimate Stress (MPa)	Tensile Modulus (GPa)	Ultimate Strain (%)
1	#1	171	859	69.1	1.2
2		199	1000	70.1	1.4
3		181	910	69.6	1.3
4		192	965	68.8	1.4
5		185	930	69.6	1.3
6		197	990	69.5	1.4
7		199	1000	69.7	1.4
8		205	1030	69.4	1.5
Average		**191**	**960**	**69.5**	**1.4**
SD		**11.3**	**56.9**	**0.4**	**0.1**
COV (%)		**5.9**	**5.9**	**0.6**	**5.8**
1	#2	196	985	69.7	1.4
2		201	1010	69.2	1.5
3		192	965	69.8	1.4
4		187	940	68.8	1.4
5		199	1000	69.2	1.4
6		191	960	69.4	1.4
7		197	990	69.7	1.4
8		201	1010	69.2	1.5
Average		**196**	**982**	**69.4**	**1.4**
SD		**5.1**	**25.5**	**0.3**	**0.04**
COV (%)		**2.6**	**2.6**	**0.5**	**2.6**
1	#3	196	985	69.1	1.4
2		193	970	68.9	1.4
3		193	970	69.2	1.4
4		190	955	70.3	1.4
5		181	910	70.4	1.3
6		200	1005	69.5	1.4
7		205	1030	69.9	1.5
8		203	1020	69.6	1.5
Average		**195**	**981**	**69.6**	**1.4**
SD		**7.7**	**38.9**	**0.5**	**0.1**
COV (%)		**4.0**	**4.0**	**0.8**	**4.3**

Lot #	Specimens	Average Tensile Capacity (MPa)	Tensile Capacity Retention R_{et}	Average Elastic Modulus (GPa)	Elastic Modulus Retention R_{et}
#1	Reference specimens	1077	89%	69.0	101%
	Conditioned specimens	960		69.5	
#2	Reference specimens	1084	91%	69.5	100%
	Conditioned specimens	982		69.4	
#3	Reference specimens	1067	92%	69.2	101%
	Conditioned specimens	981		69.6	

Mitigation against Durability Concerns

- Alkaline attack in concrete: possibly 15% loss of strength in 75 years for typical application in Ontario (worse in warm and humid climate)

  CHBDC 2014 requires Φ = 0.55 at ULS

  CHBDC 2019 requires Φ = 0.65 at ULS

- Susceptable to creep rupture for sustained load > 40% f_{pu} and fatigue resistance is around 35% f_{pu}

  CHBDC requires Φ = 0.25 at SLS

  (0.25 x 1000 MPa = 250 MPa)

- GFRP design are usually governed by SLS with a lot of reserved capacity against ULS.

Barrier Wall in Iowa

Thank you

For the most up-to-date information please visit the American Concrete Institute at:
www.concrete.org