

SEISMIC BEHAVIOR OF HIGH-STRENGTH REINFORCING STEEL AT LOW TEMPERATURES



December 2025

Prepared By:

Lina M. Espinosa Otalora, Ph.D. Candidate

Mervyn J. Kowalsky, Ph.D., P.E., Principal Investigator

Prepared For:

Alaska Department of Transportation & Public Facilities

Research, Development, and Technology Transfer

P.O. Box 112500

3132 Channel Drive

Juneau, Alaska 99811-2500

And

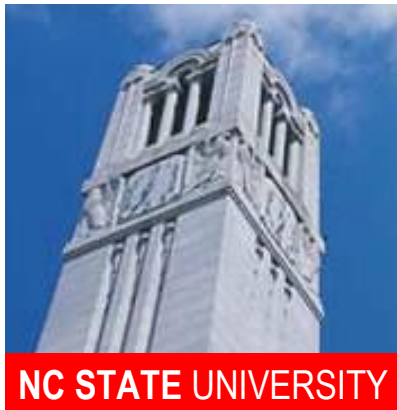
Federal Highway Administration

P.O. Box 21648

709 West 9th Street

Juneau, Alaska 99802-1648

REPORT DOCUMENTATION PAGE			Form approved OMB No.	
Public reporting for this collection of information is estimated to average 1 hour per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestion for reducing this burden to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork Reduction Project (0704-1833), Washington, DC 20503				
1. AGENCY USE ONLY (LEAVE BLANK) FHWA-AK-4000(205)		2. REPORT DATE December 2025		3. REPORT TYPE AND DATES COVERED Final Research Report January 2021 - December 2025
4. TITLE AND SUBTITLE Seismic Behavior of High-Strength Reinforcing Steel at Low Temperatures			5. FUNDING NUMBERS State Project # HFHWY00220 Federal Project # 4000(205)	
6. AUTHOR(S) Lina M. Espinosa, Ph.D. Candidate Mervyn J. Kowalsky, Ph.D., P.E.				
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) North Carolina State University Constructed Facilities Laboratory 2414 Campus Shore Drive Raleigh, NC 27695-7533			8. PERFORMING ORGANIZATION REPORT NUMBER FHWA-AK-4000(205)	
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES) Alaska Department of Transportation and Public Facilities Research, Development & Technology Transfer 3132 Channel Drive Juneau, Alaska 99811-2500			10. SPONSORING/MONITORING AGENCY REPORT NUMBER Federal Project # 4000(205)	
11. SUPPLEMENTARY NOTES				
12a. DISTRIBUTION / AVAILABILITY STATEMENT Copies available online at http://www.dot.alaska.gov/stwddes/research/search_lib.shtml			12b. DISTRIBUTION CODE N/A	
13. ABSTRACT (Maximum 200 words) Multiple regions worldwide are exposed to the combined effects of earthquakes and severe cold weather. Such is the case in Alaska, USA, where temperatures can reach sustained lows of -40°F (-40 °C) during the winter, while it is the most seismically active state in the US. Including high-strength reinforcing steel (HSRS) in members expected to form plastic hinges requires a comprehensive understanding of the material's ability to perform appropriately under these conditions. This research aims to incorporate HSRS into the seismic design of reinforced concrete bridges in cold seismic regions by addressing the impact of low temperatures on its seismic behavior. A material-level assessment was performed to evaluate the effect of temperature on the tensile properties and stress concentrations at the base of the reinforcing steel, using tension and buckled bar tension tests on ASTM A706 grade 80 and grade 100 steel. In addition, a member-level evaluation examined the effect of low temperature on the seismic behavior of RC columns reinforced with ASTM A706 grade 80 steel through quasistatic cyclic lateral loading tests. Based on experimental observations, design recommendations for incorporating high strength reinforcing steel in the seismic design of bridges in cold seismic regions were developed.				
14. KEYWORDS: Earthquake engineering, bridge columns, reinforced concrete, high strength reinforcing steel, low temperatures.			15. NUMBER OF PAGES 224	
			16. PRICE CODE N/A	
17. SECURITY CLASSIFICATION OF REPORT Unclassified	18. SECURITY CLASSIFICATION OF THIS PAGE Unclassified	19. SECURITY CLASSIFICATION OF ABSTRACT Unclassified	20. LIMITATION OF ABSTRACT None	



Constructed Facilities Laboratory
Department of Civil, Construction, and
Environmental Engineering

**SEISMIC BEHAVIOR OF HIGH-
STRENGTH REINFORCING STEEL
AT LOW TEMPERATURES**

Prepared by:

Lina M. Espinosa Otalora, *PhD. Candidate*
Dr. Mervyn Kowalsky, *Principal Investigator*

Prepared for:

Alaska Department of Transportation and Public Facilities
(AKDOT&PF)



December 2025

Constructed Facilities Laboratory
2414 Campus Shore Drive
North Carolina State University
Raleigh, NC 27695-7533
Tel: (919) 513-1733
Fax: (919) 513-1765
Email: cfl@ncsu.edu
Web Site: www.cfl.ncsu.edu

SEISMIC BEHAVIOR OF HIGH-STRENGTH REINFORCING STEEL AT LOW TEMPERATURES

Final Report for AKDOT&PF

December 2025

Notice

This document is disseminated under the sponsorship of the U.S. Department of Transportation in the interest of information exchange. The U.S. Government assumes no liability for the use of the information contained in this document.

The U.S. Government does not endorse products or manufacturers. Trademarks or manufacturers' names appear in this report only because they are considered essential to the objective of the document.

Quality Assurance Statement

The Federal Highway Administration (FHWA) provides high-quality information to serve Government, industry, and the public in a manner that promotes public understanding. Standards and policies are used to ensure and maximize the quality, objectivity, utility, and integrity of its information. FHWA periodically reviews quality issues and adjusts its programs and processes to ensure continuous quality improvement.

Author's Disclaimer

Opinions and conclusions expressed or implied in the report are those of the author. They are not necessarily those of the Alaska DOT&PF or funding agencies.

SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in ²	square inches	645.2	square millimeters	mm ²
ft ²	square feet	0.093	square meters	m ²
yd ²	square yard	0.836	square meters	m ²
ac	acres	0.405	hectares	ha
mi ²	square miles	2.59	square kilometers	km ²
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft ³	cubic feet	0.028	cubic meters	m ³
yd ³	cubic yards	0.765	cubic meters	m ³
NOTE: volumes greater than 1000 L shall be shown in m ³				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m ²	cd/m ²
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in ²	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
square meters		10.764	square feet	ft ²
meters		1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m ³	cubic meters	35.314	cubic feet	ft ³
	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m ²	candela/m ²	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in ²

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

TABLE OF CONTENTS

EXECUTIVE SUMMARY	18
CHAPTER 1. INTRODUCTION	20
1.1. High Strength Reinforcing Steel	20
1.2. Background information and research motivation	21
1.3. Research objectives	24
1.3.1. Material level objectives.....	24
1.3.2. Large-scale testing (member level) objectives	24
1.3.3. Design objectives.....	25
1.4. Document layout	25
CHAPTER 2. LITERATURE REVIEW	27
2.1. Low-temperature effect on the material properties of reinforcing and high-strength steel.....	27
2.2. Seismic behavior of reinforced concrete bridge columns at low temperatures.....	32
2.3. Temperature-related code regulations.....	35
CHAPTER 3. RESEARCH PLAN.....	37
CHAPTER 4. MATERIAL LEVEL EVALUATION	39
4.1. Definitions.....	39
4.2. Methods.....	40
4.2.1. Experimental Setup: Liquid Nitrogen Cooling System.....	41
4.2.2. Buckled Bar Tension Test	44
4.2.3. Monotonic Tension Test.....	52
4.3. Results	63
4.3.1. Buckled Bar Tension Test	63
4.3.2. Tension test results	70
4.1. Discussion	74
4.1.1. Summary of BBT Test Results.....	74
4.1.2. Summary of Tension Test Results.....	75
CHAPTER 5. MEMBER LEVEL EVALUATION	78
5.1. Experimental program.....	79
5.2. RC column specimens.....	80

5.2.1. RC columns design.....	80
5.2.2. Footing design	83
5.2.3. Construction process.....	85
5.3. Large-scale testing methods	95
5.3.1. Experimental setup	95
5.3.2. Loading protocol.....	98
5.3.3. Instrumentation.....	100
5.3.4. Material testing.....	101
Reinforcing steel:.....	101
Concrete:.....	103
5.3.5. Column predictions.....	105
5.4. Experimental response COL1: MA_RT_2IN	107
5.4.1. Summary.....	107
5.4.2. Test observations	108
5.4.3. Force-displacement response.....	115
5.4.4. Curvature Profile	117
5.5. Experimental response COL2: QST_RT_2IN	118
5.5.1. Summary.....	118
5.5.2. Test observations	119
5.5.3. Force-displacement response.....	128
5.5.4. Curvature Profile	130
5.6. Experimental response COL3: MA LT_2IN.....	131
5.6.1. Summary.....	131
5.6.2. Test observations	132
5.6.3. Force-displacement response.....	140
5.6.4. Curvature Profile	143
5.7. Experimental response COL4: QST_LT_2IN.....	144
5.7.1. Summary.....	144
5.7.2. Test observations	146
5.7.3. Force-displacement response.....	152
5.7.4. Curvature Profile	155
5.8. Experimental response COL5: MA LT_4IN.....	157
5.8.1. Summary.....	157

5.8.2. Test observations	158
5.8.3. Force-displacement response.....	166
5.8.4. Curvature Profile	169
5.9. Experimental response COL6: QST LT_4IN.....	170
5.9.1. Summary.....	170
5.9.2. Test observations	172
5.9.3. Force-displacement response.....	178
5.9.4. Curvature Profile	181
5.10. IMPACT OF LOW TEMPERATURE ON HSRS.....	183
5.10.1. Low temperature impact on the seismic behavior of RC columns reinforced with grade 80 MA steel	183
5.10.2. Low temperature impact on the seismic behavior of RC columns reinforced with grade 80 QST steel	189
5.10.3. Manufacturing process impact on the seismic behavior of HSRS	194
5.11. Summary	209
5.11.1. Low temperature impact on the seismic behavior of HSRS.....	209
5.11.2. Manufacturing process impact on the seismic behavior of HSRS at low temperatures.....	210
CHAPTER 6. CONCLUSIONS AND DESIGN RECOMMENDATIONS	212
6.1. Conclusions	212
6.1.1. Material level conclusions	212
6.1.2. Member-level conclusions.....	213
6.2. Recommendations	215
6.2.1. Material level recommendations	215
6.2.2. Member level recommendations	216
6.3. Further work.....	218
REFERENCES	220

LIST OF FIGURES

Figure 2-1 Variations of yield and tensile strength of reinforcing steel under different temperatures and strain rates. (Filiatrault & Holleran, 2001)	29
Figure 2-2 Variation in the yield and tensile strength of the reinforcing steel in a temperature range of 68°F (20°C) to -40°F (-40°C) (L. Montejo et al., 2008).....	30
Figure 2-3 Effect of cold temperature on the a) yield strength and b) tensile strength. (Levings & Sritharan, 2012)	31
Figure 2-4 Curvature profiles of the conventionally reinforced columns FL-89A and FL-89C. (L. Montejo et al., 2009a).....	34
Figure 2-5 Equivalent plastic hinge lengths for the reinforced columns FL-89A and FL-89C. (L. Montejo et al., 2009a).....	34
Figure 4-1 State of the strain of a buckled bar. (Barcley & Kowalsky, 2019)	39
Figure 4-2 Scheme of the liquid nitrogen spray cooling system.	42
Figure 4-3 Experimental setup for low-temperature buckled bar tension test.....	43
Figure 4-4 Experimental setup for low-temperature monotonic tension test.	44
Figure 4-5 Buckled bar tension test sequence.....	45
Figure 4-6 LED caps (top) and markers (bottom) on a reinforcing steel specimen.	46
Figure 4-7 Parameters used in the BBT test simplifications.....	49
Figure 4-8 Axial displacement versus bending strain. (Barcley & Kowalsky, 2019).	49
Figure 4-9 Axial displacement versus bending strain times L/D ratio for the room temperature BBT test results on Ribbed MA and QST Grade 80 (Reference from Table 4-1).....	50
Figure 4-10 Geometric specifications for the reduced cross-section machine specimens. Adapted from ASTM A370-20.....	55
Figure 4-11 #6 QST bar specimens used in the preliminary tensile tests to define the geometry of tensile test specimens. a) Full bar. b) Type 1 specimen, and c) Type 2 specimen.	56
Figure 4-12 #8 QST bar specimens used in the preliminary tensile tests to define the geometry of tensile test specimens. a) Full bar. b) Type 1 specimen, and c) Type 2 specimen.	56
Figure 4-13 Stress-strain curve using different geometries of MA Grade 80 specimens. ...	58

Figure 4-14 Stress-strain curve using different geometries of QST Grade 80 #6 specimens.	60
Figure 4-15 Stress-strain curve using different geometries of QST Grade 80 #8 specimens.	60
Figure 4-16 Distribution of the tensile properties obtained from MA and QST specimens machined to different cross-sectional areas. All values are normalized to the mean for each dataset.	61
Figure 4-17 Preliminary evaluation of conditioning cooling time on the tensile properties of HSRS.....	62
Figure 4-18 Examples of ductile(a) and brittle (b) failures obtained at room temperature BBT Test.	64
Figure 4-19 Examples of ductile (a) and brittle (b) failures obtained at low-temperature BBT Test.	64
Figure 4-20 BBT test results of ASTM A706 grade 80 QST bars.....	66
Figure 4-21 QST broken specimens tested at room (a) and low (b) temperatures. Specimens were arranged from ductile to brittle failure modes (increasing level of buckling). 66	
Figure 4-22 BBT Test results of ASTM A706 grade 80 MA bars.	67
Figure 4-23 MA broken specimens, tested at Room (a) and Low (b) temperatures. Specimens were arranged from ductile to brittle failure modes (increased level of buckling)	68
Figure 4-24 BBT Test results of smooth ASTM A706 grade 80 MA bars.....	69
Figure 4-25 MA Smooth broken specimens with ductile failures, tested at room (a) and low (b) temperatures.	69
Figure 4-26 MA Smooth broken specimens with brittle failures, tested at room (a) and low (b) temperatures.	70
Figure 4-27 Average stress-strain curves for ASTM A706 grade 80 steel at temperatures ranged from 72°F to -40°F (22°C to -40°C)	71
Figure 4-28 Stress-strain curves for ASTM A706 grade 100 steel at temperatures ranged from 72°F to -40°F (22°C to -40°C)	71
Figure 4-29 Yield strength variation as a function of temperature for ASTM A706 Grade 80 and Grade 100 reinforcing steel.	72
Figure 4-30 Ultimate strength variation as a function of temperature for ASTM A706 gr.80 and gr.100 reinforcing steel.	72

Figure 4-31 T/Y ratio variation as a function of the temperatures for ASTM A706 grade 80 and grade 100 reinforcing steel.....	73
Figure 4-32 Uniform Elongation variation as a function of temperature for ASTM A706 grade 80 and grade 100 reinforcing steel.....	73
Figure 4-33 Elastic modulus variation as a function of temperature for ASTM A706 grade 80 and grade 100 reinforcing steel.....	74
Figure 4-34 Increase in the yield strength (F_y) at various temperatures.	76
Figure 4-35 Increase in the ultimate strength (F_u) at various temperatures.	77
Figure 5-1 RC column specimen detailing (dimensions in inches)	82
Figure 5-2 Footing detailing (dimensions in inches)	84
Figure 5-3 Column cage construction.....	85
Figure 5-4 Footing cage construction and formwork	87
Figure 5-5 Footing's concrete casting	88
Figure 5-6 Placement of the specimens and scaffolding for column and cap beam construction.....	89
Figure 5-7 Thermocouples installation	90
Figure 5-8 Foam blocks used to expose reinforcing steel along the plastic hinge.	90
Figure 5-9 Formwork and frame support placement	92
Figure 5-10 Placement and sealing of cap beam steel and PVC pipes, and placement of formwork.....	92
Figure 5-11 Column and cap beam concrete casting	93
Figure 5-12 RC column specimens and removal of the formwork.....	94
Figure 5-13 Removal of the formwork	94
Figure 5-14 Cleaning of exposed reinforcing steel and column preparation for instrumentation	95
Figure 5-15 Actuator extension by Montejo et al., 2009a	96
Figure 5-16 Experimental setup (general layout).	97
Figure 5-17 Experimental setups (view inside the environmental chamber)	98

Figure 5-18 Cyclic loading protocol definition	99
Figure 5-19 Instrumentation of the RC column specimen.....	101
Figure 5-5-20 Stress-strain curve of MA bars used as longitudinal steel in the RC specimens	102
Figure 5-21 Stress-strain curve of QST bars used as longitudinal steel in the RC specimens	103
Figure 5-22 Geometric representation of the deformed RC specimen to account for P- Δ effects.....	106
Figure 5-23 P- Δ effects on the Force-Displacement predictions.....	106
Figure 5-24 The first cycle of first yielding (pulling).....	109
Figure 5-25 First cycle μ_2 (pulling). Cracks form and expand along the plastic hinge. Some hairline shear cracks were observed along the column.....	109
Figure 5-26 First cycle μ_3 (pushing): Cracks at the base of the column and along the plastic hinge expand.	110
Figure 5-27 First cycle μ_3 (pushing): Extension of cracks at the base of the column and in the footing.	110
Figure 5-28 First cycle of μ_4 (pulling).	111
Figure 5-29 First cycle of μ_4 (pulling): Concrete crushing around bar T2, and cracking and uplifting of concrete at the footing.....	111
Figure 5-30 Third cycle of μ_4 (pulling): Buckling of the top extreme bar 2T and concrete crushing.....	112
Figure 5-31 Third cycle μ_5 : Visible bar buckling in pulling cycle and fractured in pushing cycle of the top extreme bar 2T	112
Figure 5-32 First cycle μ_6 (pulling).....	113
Figure 5-33 First cycle μ_6 (pulling): Fracture of bottom extreme bar, 2B.	113
Figure 5-34 Final state of the specimen: Fracture of bars 1T, 2T, and 3T, and concrete core crushing.....	114
Figure 5-35 Damage at the bottom of the column: Fracture of bar 2B, concrete core crushing, and footing concrete breakout due to stress concentrations on footing support.....	114
Figure 5-36 Hysteretic response of Column 1: MA_RT_2in	115

Figure 5-37 Envelope of the Force-displacement response of Column 1: MA_RT_2in...	116
Figure 5-38 Low-level curvature profile of Column 1: MA_RT_2in.....	117
Figure 5-39 High-level curvature profile of Column 1: MA_RT_2in.....	118
Figure 5-40 First cycle of $\frac{1}{4} F_y$ (pulling).....	120
Figure 5-41 First cycle of $\frac{3}{4} F_y$ (pulling): Hairline cracks formed along the plastic hinge	121
Figure 5-42 First cycle μ_1 (pushing): Shear cracks formed along the plastic hinge. Some perimetral cracks were observed at the base of the column and over the plastic hinge.....	121
Figure 5-43 First cycle $\mu_{1.5}$ (pulling): Cracks elongated and expanded. Some cracks formed parallel to the reinforcing steel over the plastic hinge.....	122
Figure 5-44 Third cycle μ_2 (pulling): Cracks at the base of the column expanded, and cracks formed at the footing.	122
Figure 5-45 Third cycle μ_3 (pushing): Extension of cracks and concrete crushing.	123
Figure 5-46 First cycle of μ_4 (pulling).	123
Figure 5-47 Third cycle of μ_4 (pulling): Cracks are widening at the base of the footing.	124
Figure 5-48 Third cycle of μ_5 (pulling and pushing): Concrete crushing around the extreme bar 2T, followed by its fracture in the subsequent pushing cycle.....	124
Figure 5-49 First cycle μ_6 (pulling): buckling of top bar 3T, and fracture of bottom extreme bar 2B.....	125
Figure 5-50 First cycle of μ_6 (pushing): fracture of top bars 1T and 3T.....	125
Figure 5-51 Damage at the top of the column along the plastic hinge: Fracture of bars 1T, 2T, and 3T, and concrete core crushing.....	126
Figure 5-52 Damage at the bottom of the column: Fracture of bar 2B, concrete core crushing.....	126
Figure 5-53 Experimental extent of plasticity.	127
Figure 5-54 Hysteretic response of Column 2: QST_RT_2in	128
Figure 5-55 Envelope of the Force-displacement response of the RC column specimen #2.	129
Figure 5-56 Low-level curvature profile of Column 2: QST_RT_2in	130

Figure 5-57 High-level curvature profile of Column 2: QST_RT_2in.....	131
Figure 5-58 Temperature record of column 3:MA_LT_2in during the conditioning time and testing.....	132
Figure 5-59 Experimental setup of the low-temperature quasistatic lateral cyclic column test.....	134
Figure 5-60 First cycle of μ_1 (pushing)	134
Figure 5-61 First cycle of μ_2 (pushing). Perimetral cracks expand along the plastic hinge	135
Figure 5-62 First cycle of μ_3 (pushing). Some hairline shear cracks were observed along the column, and expansion of cracks at the base of the columns	135
Figure 5-63 First cycle of μ_4 (pulling). Initiation of concrete crushing	136
Figure 5-64 First cycle of μ_5 (pushing). Concrete damage around the longitudinal bars, damage observed in the footing, and thickening of cracks along the plastic hinge length.....	136
Figure 5-65 First cycle of μ_5 : bar 2T buckled at the first pulling cycle and fractured in the subsequent pushing cycle.....	137
Figure 5-66 First pushing cycle of μ_6 , fracture of top bar T1	137
Figure 5-67 Second cycle of μ_6 (pulling). Fracture the bottom extreme bar 2B.....	137
Figure 5-68 Damage at the top of the column along the plastic hinge: Fracture of bars 1T and 2T, and concrete core crushing.	138
Figure 5-69. Damage at the bottom of the column: Fracture of the extreme bottom bar 2B, concrete core crushing, and some footing damage.	139
Figure 5-70. Experimental extent of plasticity.	139
Figure 5-71 Hysteretic response of Column 3: MA_LT_2in.....	141
Figure 5-72 Hysteretic response of Column 3: MA_LT_2in at μ_5	141
Figure 5-73 Envelope of the Force-displacement response of the RC column specimen #3	142
Figure 5-74 Low-level curvature profile of Column 3: MA_LT_2in.....	143
Figure 5-75 High-level curvature profile of Column 3: MA_LT_2in.	144
Figure 5-76 Temperature record of column 4: QST_LT_2in during the conditioning time and testing.	145

Figure 5-77 First cycle of μ_1 (pushing)	147
Figure 5-78 First cycle of μ_2 (pushing)	147
Figure 5-79 First cycle of μ_3 (pushing) visible shear cracks and extension of existing cracks	148
Figure 5-80 Third cycle of μ_3 (pushing) widening of the perimetral cracks near the base of the column.....	148
Figure 5-81 First cycle of μ_4 (pulling): Concrete crushing	149
Figure 5-82 Spiral yielding in the third pulling cycle of μ_4 and visible bar buckling in the first pulling cycle of μ_5 – Top extreme bar 2B.....	149
Figure 5-83 Fracture of the top bars 2T and 1T in the first pushing cycle of μ_5	150
Figure 5-84 Bar buckling of the extreme bottom bar 2B at the first pushing cycle of μ_5 and fracture of the bar 2B at the third pulling cycle μ_5	150
Figure 5-85 Damage at the top of the column along the plastic hinge: Fracture of bars 1T, 2T, and 3T, and concrete core crushing.....	151
Figure 5-86 Damage at the bottom of the column along the plastic hinge: Fracture of bar 2B, and concrete core crushing	151
Figure 5-87 Experimental extent of plasticity	152
Figure 5-88 Hysteretic response of Column 4: QST_LT_2in	153
Figure 5-89 Envelope of the Force-displacement response of the RC column specimen #4	154
Figure 5-90 Low-level curvature profile of Column 4: QST_LT_2in.....	156
Figure 5-91 High-level curvature profile of Column 4: QST_LT_2in.....	156
Figure 5-92 Temperature record of column 5:MA_LT_4in during the conditioning time and testing.....	157
Figure 5-93 F_y (pulling): Hairline cracks formed along the plastic hinge in the first cycle of $3/4 F_y$ and continued expanding	160
Figure 5-94 First cycle of μ_1 (pushing): Extension of cracks	160
Figure 5-95 Third cycle of μ_2 (pushing).....	161
Figure 5-96 Third pulling cycle of μ_3 : Concrete crushing at the top of the column and cracks widening at the bottom	161

Figure 5-97 Third pulling cycle of $\mu 4$: Significant concrete crushing around the longitudinal top extreme bar, 2T.....	162
Figure 5-98 First pulling cycle of $\mu 5$: Visible buckling of top extreme bar, 2T.....	162
Figure 5-99 Second pushing cycle of $\mu 5$: Bar fracture of top extreme bar, 2T	162
Figure 5-100 Third pulling cycle of $\mu 5$: Visible buckling of adjacent bar to top bar, 3T .	163
Figure 5-101 Third pushing cycle of $\mu 5$: Bar fracture of bottom bar, 3B.....	163
Figure 5-102 Damage at the top of the column along the plastic hinge: Fracture of bars 1T, 2T, and 3T, and concrete core crushing.....	164
Figure 5-103 Brittle fracture surface of top bars (1T, 2T, and 3T).....	164
Figure 5-104 Damage at the bottom of the column along the plastic hinge: Fracture of bar 2B and concrete core crushing.....	165
Figure 5-105 Fracture of bar 2B near the base of the column	165
Figure 5-106 Experimental extent of plasticity	166
Figure 5-107 Hysteretic response of Column 5: MA_LT_4in.....	167
Figure 5-108 Envelope of the Force-displacement response of the RC column specimen #5	168
Figure 5-109 Low-level curvature profile of Column 5: MA_LT_4in.....	169
Figure 5-110 High-level curvature profile of Column 5: MA_LT_4in	170
Figure 5-111 Temperature record of column 6:QST_LT_4in during the conditioning time and testing	171
Figure 5-112 F_y (pushing). Hairline cracks formed along the plastic hinge in the first cycle of $3/4 F_y$ and continued expanding.....	173
Figure 5-113 Third pushing cycle of $\mu 1$: Extension of cracks.....	173
Figure 5-114 Third pushing cycle of $\mu 1.5$: Extension of cracks and formation of some shear cracks	174
Figure 5-115 Third pulling cycle of $\mu 3$: Concrete flaking near the base of the column....	174
Figure 5-116 First pulling cycle of $\mu 4$: Concrete crushing near the base of the column...	175
Figure 5-117 Third pulling cycle of $\mu 4$: Buckling of top extreme bar 2T	175
Figure 5-118 First pulling cycle of $\mu 5$: Bar fracture of bottom extreme bar 2B	176

Figure 5-119 First pushing cycle of $\mu 5$: Bar fracture of top extreme bar 2T.....	176
Figure 5-120 Second pulling cycle of $\mu 5$: Bar fracture of adjacent bar to top extreme bar 1B	176
Figure 5-121 Damage at the top of the column along the plastic hinge: Fracture of bar 2T and concrete core crushing.....	177
Figure 5-122 Damage at the bottom of the column along the plastic hinge: Fracture of bars 2B and 1B, and concrete core crushing	177
Figure 5-123 Experimental extent of plasticity.	178
Figure 5-124 Hysteretic response of Column 6: QST_LT_4in	179
Figure 5-125 Envelope of the Force-displacement response of the RC column specimen #6	180
Figure 5-126. Low-level curvature profile of Column 6: QST_LT_4in.....	182
Figure 5-127. High-level curvature profile of Column 6: QST_LT_4in.....	182
Figure 5-128 Comparison of the hysteretic response of Columns 1 and 3: MA_RT_2in vs. MA_LT_2in	184
Figure 5-129 Comparison of the first cycle envelope of the force displacement response of Columns 1 and 3: MA_RT_2in vs. MA_LT_2in.....	185
Figure 5-130 Comparison of the third cycle envelope of the force displacement response of Columns 1 and 3: MA_RT_2in vs. MA_LT_2in.....	185
Figure 5-131 Visible buckling in the top extreme bar for the MA columns tested at room and low temperatures	186
Figure 5-132 Top extreme bar fractures for the MA columns tested at room and low temperatures	186
Figure 5-133 Concentration of concrete crushing at the base of the columns.....	187
Figure 5-134 Comparison of the curvature distribution at $\mu 1$ of Columns 1 and 3: MA_RT_2in vs. MA_LT_2in.....	188
Figure 5-135 Comparison of the curvature distribution at $\mu 4$ of Columns 1 and 3: MA_RT_2in vs. MA_LT_2in.....	188
Figure 5-136 Comparison of the hysteretic response of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in.....	190

Figure 5-137 Comparison of the first cycle envelope of the force displacement response of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in	190
Figure 5-138 Comparison of the third cycle envelope of the force displacement response of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in.	191
Figure 5-139 Buckling of the top extreme bar in the QST columns, tested at room and low temperatures.	191
Figure 5-140 Fracture of the top extreme bar in the QST columns, tested at room and low temperatures.	191
Figure 5-141 QST broken bars subjected to the BBT test at low temperature. Specimens were arranged from ductile to brittle failure modes (increasing levels of buckling at the top).	192
Figure 5-142 Comparison of the curvature distribution at μ_1 of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in.....	193
Figure 5-143 Comparison of the curvature distribution at μ_4 of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in.....	193
Figure 5-144 Concentration of concrete crushing at the base of the columns.....	194
Figure 5-145 Comparison of the hysteretic response of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in.....	195
Figure 5-146 Comparison of the first cycle envelope of the force displacement response of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in.	196
Figure 5-147 Comparison of the third cycle envelope of the force displacement response of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in.	196
Figure 5-148 Comparison of the MA and QST columns tested at room temperature, μ_5^{+1} : buckling in the MA specimen, and significant crushing along the top extreme bar in the QST specimen.	197
Figure 5-149 Comparison of the top bar fractures on the MA and QST columns tested at room temperature.	197
Figure 5-150 Concentration of concrete crushing at the base of the columns.....	197
Figure 5-151 Comparison of the curvature distribution at μ_1 of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in.....	198
Figure 5-152 Comparison of the curvature distribution at μ_4 of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in.....	199

Figure 5-153 Comparison of the hysteretic response of Columns 3 and 4: MA_LT_2in vs. QST_LT_2in.....	200
Figure 5-154 Comparison of the first cycle envelope of the force displacement response of Columns 3 and 4: MA_LT_2in vs. QST_LT_2in.....	201
Figure 5-155 Comparison of the third cycle envelope of the force displacement response of Columns 3 and 4: MA_LT_2in vs. QST_LT_2in.....	201
Figure 5-156 Top extreme bar buckling for the MA and QST columns with 2-inch spiral spacing tested at low temperatures.	202
Figure 5-157 Top extreme bar fractures for the MA and QST columns with 2-inch spiral spacing tested at low temperatures	202
Figure 5-158 Comparison of the curvature distribution at μ_1 of Columns 3 and 4: MA_LT_2in vs. QST_LT_2in.....	203
Figure 5-159 Comparison of the curvature distribution at μ_4 of Columns 3 and 4: MA_LT_2in vs. QST_LT_2in.....	203
Figure 5-160 Comparison of the hysteretic response of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in.	205
Figure 5-161 Comparison of the first cycle envelope of the force displacement response of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in.....	205
Figure 5-162 Comparison of the third cycle envelope of the force displacement response of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in.....	206
Figure 5-163 Buckling of the top extreme top bars of specimens 5 and 6	206
Figure 5-164 Fracture of the top extreme bars of specimens 5 and 6.....	206
Figure 5-165 Brittle fracture of the MA and QST longitudinal steel.	207
Figure 5-166 Comparison of the curvature distribution at μ_1 of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in.....	208
Figure 5-167 Comparison of the curvature distribution at μ_4 of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in.....	208

LIST OF TABLES

Table 2-1 Low-temperature-based requirements for earthquake-resisting members.(Alaska Bridges and Structures Manual, 2017).....	36
Table 3-1 Research approach overview.....	38
Table 4-1 Material-level experimental program.	41
Table 4-2 Evaluation goals of the material-level experimental program.	41
Table 4-3 Cooling system components.....	42
Table 4-4 Empirical equation developed for low-temperature BBT tests.	50
Table 4-5 Test matrix for the BBT test program.	51
Table 4-6 Test matrix for selection of tension test specimens.....	55
Table 4-7 Tensile properties comparison between Type 1 and Unaltered MA specimens.	57
Table 4-8 Tensile properties comparison between Type 1 and Unaltered QST specimens.	59
Table 4-9 Tensile properties comparison between Type 2 and Unaltered QST specimens.	59
Table 4-10 Tension test matrix.	61
Table 4-11 Results of the preliminary evaluation of using different cooling times.	63
Table 4-12 Critical bending strain on ASTM 706-22 Grade 80 MA, and QST ribbed bars, and MA smooth bars.....	75
Table 5-1 Large-scale testing test matrix.....	79
Table 5-2 Design Material Properties (Seismic Design Criteria Version 2.0, 2019)	81
Table 5-3 Load protocol definition - Force control range	99
Table 5-4 Calculation of equivalent yield displacement.....	100
Table 5-5 Load protocol definition - Displacement control range.	100
Table 5-6 Tensile properties of the longitudinal steel	103
Table 5-7 Tensile properties of the transverse steel.....	103
Table 5-8 Concrete compressive strength of the RC column specimens.....	104
Table 5-9 Summary of column parameters.....	107

Table 5-10 Column 1 damage progression summary.	108
Table 5-11 Summary of column parameters.....	119
Table 5-12 Column 2 damage progression summary	120
Table 5-13 Summary of column parameters.....	132
Table 5-14 Column 3 damage progression summary	133
Table 5-15 Summary of column parameters.....	145
Table 5-16 Column 4 damage progression summary	146
Table 5-17 Summary of column parameters.....	158
Table 5-18 Column 5 damage progression summary	159
Table 5-19 Summary of column parameters.....	171
Table 5-20 Column 6 damage progression summary	172
Table 6-1 Low temperature strength increase factors for grade 80 steel	215

EXECUTIVE SUMMARY

Multiple regions worldwide are exposed to the combined effects of earthquakes and severe cold weather. Such is the case in Alaska, USA, where temperatures can reach sustained lows of -40°F (-40°C) during the winter, while it is the most seismically active state in the US. Including high-strength reinforcing steel in members expected to form plastic hinges (i.e., bridge piers) requires a comprehensive understanding of the material's ability to perform appropriately under these conditions. The primary concern is how temperature affects stress concentration at the base of the reinforcing steel's deformations and how different steel properties contribute to this effect. Aggravating the impact of stress concentrations at the base of the reinforcing steel ribs can ultimately reduce the seismic behavior of reinforced concrete structures. This research aims to incorporate high-strength reinforcing steel into the seismic design of reinforced concrete bridges in cold seismic regions by assessing its seismic behavior at low temperatures, accounting for factors such as manufacturing processes and steel grades.

This research report presents an overview of motivation, research goals, methods, experimental results, and recommendations on using HSRS in cold seismic regions. A literature review was conducted to understand existing knowledge and identify research gaps regarding the effects of low temperatures on the material properties of reinforcing and high-strength steel, the low-temperature impact on the seismic behavior of reinforced concrete bridge columns, and the available code regulations that account for temperature in the seismic design of bridges. This research had a strong experimental component, divided into a material-level evaluation and a member-level evaluation. The material-level assessment focused on the impact of temperature on the tensile properties of high-strength reinforcing steel and on stress concentrations at the base of the reinforcing steel, using tension and

buckled bar tension tests on ASTM A706-22 grade 80 and grade 100 steel. The member-level evaluation addressed the low-temperature effect on the seismic behavior of RC columns reinforced with ASTM A706 grade 80 steel through quasistatic cyclic lateral loading tests. Based on experimental observations, design recommendations for incorporating high-strength reinforcing steel into the seismic design of bridges in cold seismic regions were developed.

CHAPTER 1. INTRODUCTION

1.1. High Strength Reinforcing Steel

In this study, "high-strength reinforcing steel" (HSRS) refers to reinforcing steel with a specified yield strength of 80 ksi (550 MPa) or greater. Two main manufacturing processes are available in the U.S. to produce HSRS: micro-alloying (MA) and quench-self-tempering (QST). Micro-alloying involves adding small quantities of alloying elements to the steel-making process. The most common alloying elements include titanium (Ti), niobium (Nb), and vanadium (V). The proportions per alloy are typically 0.02-0.15% by weight of the total steel composition. Micro-alloy additions lead to grain refinement and the precipitation of nano-sized particles that restrict dislocation motion, the primary mechanism of plastic deformation. Thus, the steel's yielding is delayed by limiting the mechanism that leads to plastic deformation. (NIST, 2014; UK CARES, 2011). MA grade 80 bars can be manufactured to meet the requirements of ASTM A706/706M-22.

In the quenched self-tempering process, water is sprayed rapidly to cool (quench) the bars' surfaces after they leave the last rolling pass. The cooled outer layer undergoes a martensitic transformation, in which heat from the warmer core tempers the outer shell of the steel, increasing ductility and reducing brittleness in the hardened outer shell (self-tempering). The slow cooling of the core generates a relatively lower-strength, more ductile interior microstructure. The mechanical properties of QST steel result from the combination of the bars' cross-sectional characteristics, including a tough exterior shell and a ductile core (NIST, 2014; UK CARES, 2011). While the QST process is cheaper than the MA process, the T/Y ratios tend to be near the lower limit required by ASTM A706/706M-22 (1.25). However, QST bars with higher T/Y ratios are now available, which are obtained from small

additions of micro-alloying elements (J. Selzer, Director of quality assurance and process improvement at CMC, personal communication, 2024)

1.2. Background information and research motivation

ASTM A706 grade 60 steel is widely used for seismic applications in the United States, as this type of steel exhibits material properties (uniform elongation, T/Y ratio) that result in sufficient ductility and energy dissipation when placed in reinforced concrete members. Extensive structural testing and implementation of ASTM A706 grade 60 steel in codes have facilitated broad application in design. Nonetheless, HSRS is gaining popularity in the construction industry due to mills' ability to produce and expand the availability of higher-grade reinforcing bars, as well as the benefits of its use.

HSRS provides construction, cost, and environmental benefits to structures. These benefits are attained by reducing the reinforcing steel required to achieve the necessary sectional strength. HSRS helps alleviate steel congestion, often found in joints of seismic-resistant structures, and reduces labor time and material transportation. Overcoming the challenges associated with using this material helps provide engineers with additional tools to develop more sustainable and cost-effective solutions while ensuring safe and reliable structural designs.

In 2009, ASTM A706/706M-09 included chemical and mechanical specifications for Grade 80 reinforcing steel. (ASTM International, 2009). These specifications for Grade 80 were adopted in 2011 in the Guide Specifications for LRFD seismic bridge design (2011) and ACI318-11(AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011; ACI Committee 318, 2011). However, these standards limit grade 80 steel to capacity-protected members due to uncertainties regarding seismic behavior.

For the first time, ACI318-19 included grade 80 steel for special moment frames, based on research showing comparable seismic performance to members reinforced with grade 60 steel. Grade 100 steel remains limited to capacity-protected members. This step into the inclusion of HSRS in the seismic design of structures hasn't been taken in the bridge design codes, and the use of HSRS is not yet allowed for members expected to form plastic hinges. However, the California Department of Transportation has assembled a 'Grade 80 Implementation Team' to introduce the use of HSRS for plastic hinges, pending the outcome of current research.

Research conducted through large-scale tests and computational modeling of the cyclic response of reinforced concrete (RC) has increased confidence in the performance of HSRS for members expected to form plastic hinges. (Barbosa et al., 2016; Barcley & Kowalsky, 2019a; Li et al., 2018; Manhard & Kowalsky, 2019; Rautenberg et al., 2012; Sokoli et al., 2020; Sokoli & Ghannoum, 2016). Plastic hinge-forming members with HSRS have shown displacement capacity similar to that of RC members reinforced with ASTM A706 grade 60 steel. On the other hand, the energy dissipation capacity of RC members using HSRS has been shown to decrease. Nonetheless, some authors relate this behavior to lower stiffness due to decreased steel area (Barbosa et al., 2016; Trejo et al., 2014).

The predominant failure in well-detailed columns is bar buckling, followed by bar fracture, which causes significant strength degradation (Barcley & Kowalsky, 2019a; Manhard & Kowalsky, 2019). Buckling is a principal factor in bar fracture due to the compressive strains generated along the bar cross-section from buckling-induced bending. Stress concentrations at the base of the steel's deformations can lead to microcracks, initiating bar fracture in the subsequent tensile cycle. (Restrepo-Posada et al., 1994; Sokoli & Ghannoum, 2016).

Research conducted by Barclay & Kowalsky (2019a, 2020), Manhard & Kowalsky (2019), and Thangjitham & Kowalsky (2023) have shown that the reinforcing steel's ability to sustain buckling-induced bending strain (BIBS) is influenced by factors that affect strain concentrations at the base of the deformations, such as the chemical composition and manufacturing process of the reinforcing steel, the ratio of rib height to rib radius at the base of the deformations, and the manufacturing processes. Reinforcing steel with reduced ability to resist BIBS may reduce the displacement ductility of members reinforced with it.

The information available on the seismic behavior of HSRS is limited to room temperature conditions. However, regions around the world, including the Northwest United States, Northwest Canada, Northeast Russia, northern China, and Japan, are exposed to low temperatures and earthquakes. In the United States, for example, Alaska stands out as the most seismically active state and a region where winters routinely achieve sustained temperatures as low as -40°F (-40°C).

The primary concern regarding low temperatures is the potential impact on existing strain concentrations at the base of the ribs, which could lead to earlier fracture than in conventional steel and further reduce displacement capacity. Paving the way for using HSRS in cold seismic regions requires a comprehensive understanding of the effect of low temperature on steel's material properties and the strain demand and capacity of HSRS in RC members.

1.3. Research objectives

This research aims to understand the impact of low temperatures on the seismic behavior of HSRS, develop design recommendations, and incorporate HSRS into bridge design in cold seismic regions. To accomplish the research goals, a material-level evaluation and large-scale testing (member-level) were conducted to develop design recommendations. The objectives of these phases are summarized below.

1.3.1. Material level objectives

1. Address the low-temperature effect on the tensile properties of ASTM A706 grade 80 steel and grade 100 steel using monotonic tension tests on reinforcing steel bars at temperatures varying from 72°F to -40°F (22 °C to -40°C).

2. Evaluate the low-temperature effect on the critical bending strain of ASTM A706 grade 80 and grade 100 steel, by conducting buckled bar tension tests (BBT Test) at 72°F and -40°F (22 °C and -40°C).

1.3.2. Large-scale testing (member level) objectives

1. Evaluate the impact of the temperature on the seismic behavior of RC columns reinforced with MA and QST ASTM A706 grade 80 steel by large-scale quasi-static lateral cyclic loading tests varying from 72°F to -40°F (22 °C to -40°C). Specifically, address how different manufacturing processes and temperatures affect the global and local responses of the RC members, including the force-displacement response, the spread of plasticity, the strain profiles, the strain limit states associated with damage progression, and the base curvatures.

2. Evaluate the impact of the manufacturing process of the reinforcing steel (MA and QST ASTM A706 grade 80 steel) on the seismic behavior of RC columns tested at low

temperatures and its susceptibility to brittleness under different confinement levels (using two ratios of transverse steel).

1.3.3. Design objectives

1. Develop recommendations to account for the impact of the temperature on the mechanical properties of HSRS for structural analysis and seismic design of bridge columns using HSRS, such as plastic hinge reduction factors, expected material properties, and preferred manufacturing process (if any).

1.4. Document layout

Chapter 1 introduces the research report, including the definition of HSRS, the research background and motivation, and the research objectives; finally, the chapter explains the scope of the document.

Chapter 2 summarizes the main findings obtained from the literature review on the effect of low temperature on the seismic behavior of HSRS. This chapter includes information and research gaps on the impact of low temperatures on the material properties of high-strength and reinforcing steel, on the seismic behavior of RC columns reinforced with ASTM A706 grade 60 steel, and on temperature-based code requirements for the seismic design of RC structures.

Chapter 3 presents an overview of the research plan proposed to address the seismic behavior of HSRS at low temperatures. The project is divided into two experimental phases: material-level evaluation, large-scale testing (member-level), and the development of design recommendations. These phases are introduced and explained in the chapter, and the details of each phase are expanded in the corresponding chapter.

Chapter 4 explains in detail the methods followed for the material-level evaluation of HSRS. It also presents experimental programs, setups, and specimen definitions. In addition, results on the BBT and Tension tests are presented and analyzed in this chapter.

Chapter 5 presents the methods for the member-level evaluation conducted to address the impact of temperature on the seismic behavior of RC columns using HSRS. This chapter contains the experimental program, a description of the RC columns specimens and construction process, the testing methods: experimental setup, instrumentation, test predictions, and loading protocol, and the experimental observations of the 6 columns tested. This chapter concludes with a discussion of the observed impact of low temperature on HSRS and the influence of manufacturing processes on the low-temperature seismic behavior of RC columns tested with HSRS.

Chapter 6 presents the conclusions from the experimental observations in this study and recommendations for incorporating HSRS into the seismic design of bridges exposed to the combined action of earthquakes and low temperatures.

CHAPTER 2. LITERATURE REVIEW

A literature review was conducted to identify the research gaps and existing knowledge on the seismic behavior of HSRS at low temperatures. It was found that this subject was not directly addressed in the available literature. However, information was collected on the low-temperature effects on the material properties of reinforcing and high-strength steel, the low-temperature impact on the seismic behavior of reinforced concrete bridge columns using ASTM A706 grade 60 steel, and the available code regulations for accounting for temperature in the seismic design of bridges. An extensive literature review report was submitted to AKDOT&PF in October 2022 (Otálora & Kowalsky, 2022); therefore, this chapter summarizes the main findings from the literature review report. Further details can be found in the literature review report.

2.1. Low-temperature effect on the material properties of reinforcing and high-strength steel

Past research at the material scale focused on the effect of the temperature on the tensile properties (i.e., yield, tensile strength, modulus of elasticity, and elongation capacity), impact toughness, and brittleness of high-strength steel and mild reinforcing steels. Previous studies (Elices et al., 1986; Filiatrault & Holleran, 2001; Kunitake et al., 1979; Levings & Sritharan, 2012; Liu et al., 2014; Perez-Martin et al., 2019; Xie et al., 2018; Yan et al., 2014, 2021; Yan & Xie, 2017) showed that high-strength steel and mild reinforcing steels presented a gradual increment of the yield and tensile strength as the temperature dropped. In contrast, the modulus of elasticity and elongation capacity were minimally correlated with temperature.

Among the more relevant studies to this research, Levings & Sritharan, (2012), L. Montejó et al. (2009a), and Sloan et al. (2005) evaluated the effects of the cold temperature on the properties of ASTM A706 grade 60 reinforcing steel for applications in cold seismic regions.

Sloan et al. (2005) conducted tensile test on ASTM A706 grade 60 bars using approximate temperatures of -4°F (-20°C) and 68°F (20°C). The effect of strain rates on the tensile response of the steel was evaluated using earthquake-type strain rates, ranging from 0.01 s^{-1} to 0.001 s^{-1} . In this study, the specimens had a diameter of $5/8$ in and a length of 4 in between grips. This research was conducted without temperature control. The specimens were cooled using dry ice and subsequently tested. Therefore, the specimens' temperature increased during the tests because they were exposed to room temperature.

(Sloan et al., (2005) reported that the yield and tensile strength increased by 8% and 10%, from 68°F (20°C) to -4°F (-20°C), with no impact on the elastic modulus. The elongation at maximum stress (uniform elongation) of the cold specimens decreased by 7.9% at low temperatures in tests conducted at a strain rate of 0.01 s^{-1} . At lower strain rates, the uniform elongation was not significantly affected. This may have been a consequence of the specimens warming up with the slower strain rate.

Similarly, Filiatrault & Holleran, (2001) studied the combined effects of characteristic seismic strain rates and low temperature on the tensile properties of reinforcing steel with 58 ksi yield strength. They evaluated the uniaxial response of reinforcing steel coupons at quasi-static and seismic strain rates (up to 0.1 s^{-1}) and temperatures of 68°F (20°C), -4°F (-20°C), and -40°F (-40°C). The steel coupons had a diameter of 0.24 in and were all machined from a single 39 ft. long deformed bar with a nominal diameter of $5/8$ in.

Filiatrault & Holleran, (2001) observed that at low temperatures and high strain rates, the yield strength and tensile strength of the reinforcing steel increase, as shown in Figure 2-1. The yield strength and tensile strength tested at a strain rate of 0.1s^{-1} and a temperature of -40°F (-40°C) increased by about 20% and 10%, respectively, compared with the specimens tested at a quasi-static strain rate (0.000080 s^{-1}) and room temperature 68°F (20°C). Filiatrault & Holleran, (2001) Also observed that low temperatures might increase the effect of the strain rate on the yield strength, as can be noticed in Figure 2-1, in which the trend line slope between the yield strength and the strain rates increases as the temperature decreases. The temperatures and strain rates did not significantly affect the elastic modulus and the elongation at the maximum strength.

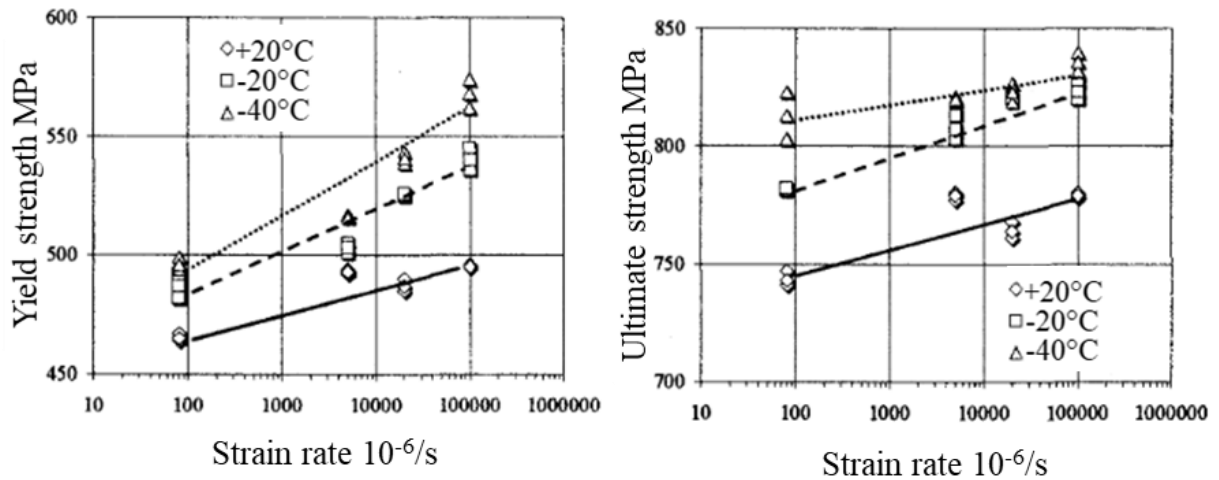


Figure 2-1 Variations of yield and tensile strength of reinforcing steel under different temperatures and strain rates. (Filiatrault & Holleran, 2001)

Montejo et al. (2008) represent the temperature effect on the yield and tensile strength using the results obtained from Filiatrault & Holleran, (2001) and Sloan et al., (2005) considering a 10% strength increase from -4°F (-20°C) to -40°F (-40°C), with a linear variation from 0°F (32°C) to -4°F (-20°C). This model is presented in Figure 2-2.

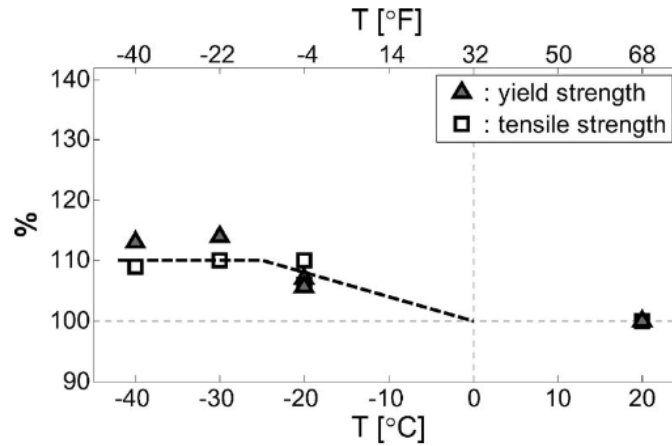


Figure 2-2 Variation in the yield and tensile strength of the reinforcing steel in a temperature range of 68°F (20°C) to -40°F (-40°C) (L. Montejo et al., 2008)

Furthermore, Levings & Sritharan (2012) studied the effect of low temperatures and strain rates on the tensile properties of ASTM A706 grade 60 reinforcing steel. The properties evaluated were yield and tensile strength, elastic modulus, and elongation at fracture, according to ASTM E8/E8M. The gauge length for the measurement of the elongation at fracture was not specified. Tensile tests were conducted on 3ft long deformed bars with nominal diameters of 6/8 in and 1 in. The temperatures and strain ranges evaluated ranged from 68°F (20°C) to -40°F (-40°C) and 0.003 m^{-1} to 0.3 m^{-1} (0.00005 s^{-1} to 0.005 s^{-1}), respectively.

At a quasistatic strain rate (0.003 m^{-1}), the yield and tensile strength of the ASTM A706 grade 60 increased by 5.1% and 6.3%, respectively, from 68°F (20°C) to -40°F (-40°C). Levings & Sritharan, (2012) noticed that the yield and tensile strength increased progressively as the temperature decreased. They observed higher yield and tensile strengths even at temperatures higher than 32°F (0°C). Levings & Sritharan, (2012) represented the temperature effect on the yield and tensile strength in a quadratic function (Figure 2-3), which differs from the trend presented by Montejo et al., (2008) in Figure 2-2.

Levings & Sritharan (2012) observed that higher strain rates increased yield strength and tensile strength. Changing the strain rate from 0.003 min^{-1} to 0.3 min^{-1} increased the yield and tensile strength by 3% and 1.67%, respectively, at -4°F (-20°C). Higher strain rates seem to increase the effect of the low temperatures on the yield and tensile strength (as observed by Filiatrault & Holleran (2001)), leading to higher values. The elastic modulus and the elongation at fracture remain almost constant under both temperature and strain-rate effects.

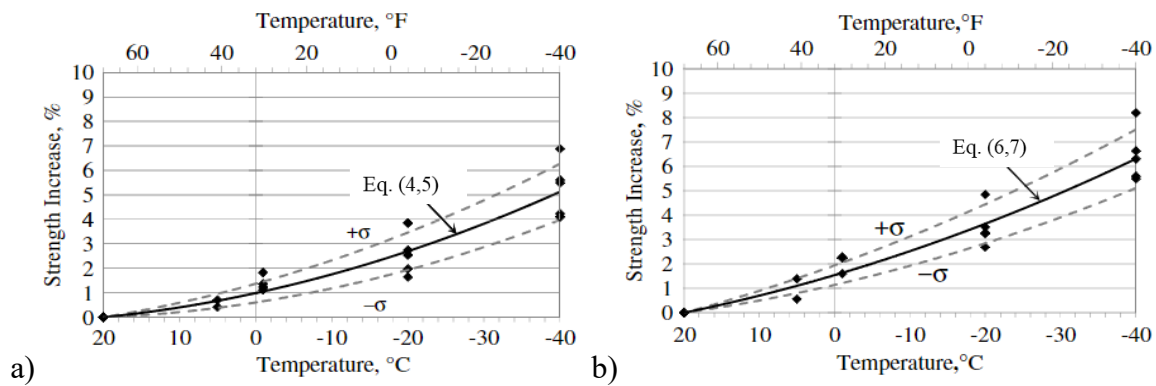


Figure 2-3 Effect of cold temperature on the a) yield strength and b) tensile strength. (Levings & Sritharan, 2012)

Elices et al., (1986), and Kunitake et al., (1979) also evaluated the effect of bar geometry and manufacturing processes on reinforcing steel's tensile properties. Stress concentrations in ribbed bars and bars with superficial imperfections (notches) were shown to decrease the strength of the bars as the temperature dropped. In addition, the manufacturing process significantly influenced the strength and elongation capacity of reinforcing steel at low temperatures. However, these studies were conducted in the 1980s using specific manufacturing methods that do not reflect current manufacturing practices (MA and QST).

On the other hand, several Charpy V-notch tests have been performed on high-strength and reinforcing steel to evaluate their impact toughness at low temperatures. For example, Hyvonen (1985), Liu et al., (2014), Mori et al., (1984), Morimoto et al., (1982), Rodriguez-Martin et al., (2007), Tong et al., (2018) showed that the high-strength and reinforcing steel's energy absorption capacity decreases with decreasing temperature. In addition, microstructural studies of the fracture surfaces of Charpy V-notch specimens showed a shift in failure mode from ductile to brittle as temperature decreased; however, this tendency was not evident in the tension tests.

One of the biggest concerns when using high-strength reinforcing steel at low temperatures is the contribution of stress concentrations to the bars' brittleness. Most of the studies performed on high-strength steel and reinforcing steel reported a predominance in ductile failures within our interest range (until -40°F, -40°C) (Elices et al., 1986; Filiatrault & Holleran, 2001; Levings & Sritharan, 2012; Liu et al., 2014; Perez-Martin et al., 2019; Yan et al., 2014, 2021; Yan & Xie, 2017). These studies analyzed the failure modes of bars subjected to pure tension. Research on the buckling behavior of reinforcing steel subjected to low temperatures was not identified. Understanding this topic is essential because buckling is a principal failure mechanism in seismic-resistant reinforced concrete members. (Barclay & Kowalsky, 2019b; Restrepo-Posada et al., 1994).

2.2. Seismic behavior of reinforced concrete bridge columns at low temperatures

To the author's knowledge, no studies have examined the seismic behavior of high-strength steel reinforced concrete columns at low temperatures, and few have evaluated the lateral cyclic loading behavior of large-scale reinforced concrete columns at low temperatures. For example, Sritharan et al. (2007) addressed the effect of subfreezing

temperatures on soil-structure interaction. They found that frozen soil conditions relocate the plastic hinge and decrease the plastic hinge length in cast-in-drilled-hole (CIDH) foundation shafts, decreasing lateral displacement capacity. In addition, a research program was conducted at North Carolina State University to assess the impact of low temperature on the seismic behavior of reinforced concrete columns using ASTM A706 grade 60 steel. (L. Montejo et al., 2009b, 2009a; Sloan et al., 2005).

The studies conducted by Montejo et al. (2009b, 2009a); and Sloan et al. (2005) included the quasistatic lateral loading test of seven pairs of RC columns at room temperature and low temperature, including shear and flexurally dominant RC columns and RCFST columns. These studies showed that RC columns tested at temperatures of -40°F (-40°C) and -4°F (-20°C) exhibited higher strength and reduced spread of plasticity than those tested at room temperature (72°F (22°C)).

Sloan (2005) showed that the drop in the temperature increased the flexural strength by 7.9% and 13% at temperatures of -4°F (-20°C) and -40°F (-40°C), respectively. In addition, the column displacement capacity was also affected by low temperatures. The column drift, before the steel fracture, decreased from 5.97% at 74°F (23°C) to 5.39% and 5.32% at -4°F (-20°C) and -40°F (-40°C), respectively. Similarly, Montejo et al. (2009b) observed that the flexural strength increased by 14% in the cold specimen compared with specimens tested at room temperature. However, he did not observe that subfreezing conditions significantly affected the columns' displacement capacity. Nonetheless, the failure of the cold specimen appeared more brittle.

Although the increase in the flexural strength led to an increase in the shear demand of the element when subjected to cold temperatures, Montejo et al. (2009b) showed that the shear capacity of the columns increased at a higher rate, overcoming the increased shear

demand. Therefore, shear failures were not identified as a concern for well-detailed columns subjected to the combined effects of earthquakes and low temperatures.

Another important observation was that the spread of plasticity decreased with lower temperatures, as higher curvatures were concentrated over a smaller distance from the base (Figure 2-4). As a result, the equivalent plastic hinge length was 36% lower than the one obtained at room temperature. Figure 2-5 presents the change in the equivalent plastic hinge length as a function of temperature for different displacement ductility levels for the flexurally dominant specimens tested by Montejo et al., 2009b .

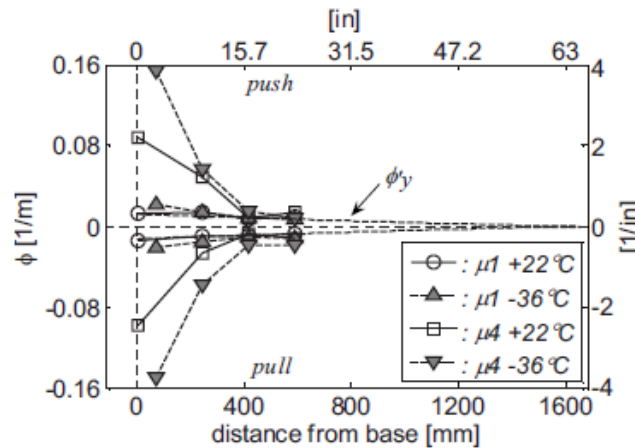


Figure 2-4 Curvature profiles of the conventionally reinforced columns FL-89A and FL-89C. (L. Montejo et al., 2009a)

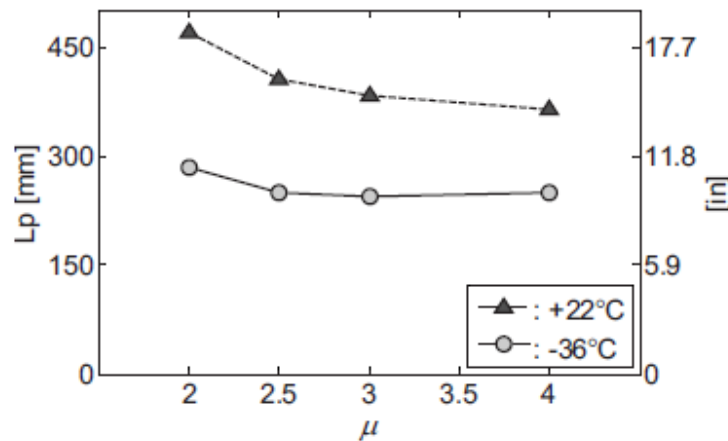


Figure 2-5 Equivalent plastic hinge lengths for the reinforced columns FL-89A and FL-89C. (L. Montejo et al., 2009a)

Moreover, Montejo et al. (2009b, 2009a), Montejo et al. (2008), and Sloan et al. (2005) observed that low temperatures did not affect the energy dissipation capacity of the members reinforced with ASTM A706 grade 60 steel. However, prior research has shown that high-strength reinforcing steel reduces energy dissipation at room temperatures. (Barbosa et al., 2016; Barclay et al., 2019; Manhard & Kowalsky, 2019; Rautenberg et al., 2012; Trejo et al., 2014). This indicates that further research is needed to analyze whether the lower temperature aggravates the change in the energy capacity of high-strength reinforced columns.

Reductions in displacement capacity due to stress concentration at the base of the reinforcing steel's ribs, the spread of plasticity, and the energy dissipation capacity are among the main concerns to be addressed in the structural-level assessment of the seismic behavior of HSRS at low temperatures.

2.3. Temperature-related code regulations

National seismic design codes have not directly accounted for the effects of low temperatures on structures' seismic behavior, resulting in a lack of guidance on design practices in cold seismic regions. As a result of research and years of experience, Alaska DOT&PF developed the Alaska Bridges and Structures Manual to provide design guidelines that bridges and structures under the responsibility of Alaska's DOT&PF (Alaska Bridges and Structures Manual, 2017).

The design procedures in the manual are displacement-based; therefore, special care is given to estimating material properties to predict the structure's behavior accurately. Table 2-1 presents low-temperature strength factors that should be applied to the expected material properties and modifications to the analytical plastic hinge length (Alaska Bridges and

Structures Manual, 2017). These requirements were provided by Montejo et al. (2010), who conducted pushover analyses and calibrated fiber-based lumped plasticity models with experimental results to develop the design recommendations, and should be applied to structures in regions where temperatures below -4°F (-20°C) are reached. These modification factors are limited to ASTM A706 grade 60 steel, and further research is needed to develop design recommendations incorporating HSRS in cold seismic regions.

The manual also specifies in Section 17.6.4 that frozen soil stiffness should be included in the lateral analysis of foundations and substructures. Seasonal soil freezing has been shown to alter the fixation conditions between the soil and deep foundations, leading to plastic hinge relocation and affecting the seismic behavior of bridges (Alaska Bridges and Structures Manual, 2017).

Table 2-1 Low-temperature-based requirements for earthquake-resisting members.(Alaska Bridges and Structures Manual, 2017)

Properties	Low temperature-based modifications	Section
f_{ce} = expected concrete compressive strength	$f_{ce-cold} = 1.4 \times f_{ce}$	18.3
f_{ye} = expected yield stress of steel	$f_{ye-cold} = 1.1 \times f_{ye}$	18.3
f_{ue} = expected tensile strength of steel	$f_{ue-cold} = 1.1 \times f_{ue}$	18.3
L_p = analytical plastic hinge length	$L_{p-cold} = 0.6 \times L_p$	18.3

CHAPTER 3. RESEARCH PLAN

The experimental program includes material-level and large-scale evaluations to understand how temperature affects the seismic behavior of HSRS. It investigates whether parameters help the steel maintain better performance at low temperatures (i.e., one manufacturing process over the other) and to what extent the material and member-level properties change.

Three phases are proposed for this study: 1) A material-level characterization of the HSRS at temperatures ranging from 72°F to -40°F (22°C to -40°C) employing buckled bar tension tests and monotonic tension tests. 2) A member-level characterization of the seismic response of HSRS at room and low temperatures through large-scale quasi-static cyclic lateral loading tests on RC columns. 3) Develop design recommendations to incorporate HSRS in cold seismic regions using the information obtained from this research's experimental and analytical phases. The detailed methods proposed per phase will be expanded in the following chapters. Table 3-1 presents a general scheme of the methods followed in each phase.

In the experimental phase of this study, HSRS from two of the largest reinforcing steel producers in the US is evaluated at temperatures ranging from 72°F to -40°F (22°C to -40°C). The experiments include ASTM A706 grade 80 steel manufactured by NUCOR, which primarily produces MA reinforcing steel, and CMC, which leads the QST steel production. ASTM A706 grade 100, made by NUCOR, is also evaluated at the material level.

Table 3-1 Research approach overview.

		BBT Test		Monotonic Tension Test	
1. Material level evaluation		Specimens: ASTM A706 Grade 80 Properties to evaluate: Critical bending strain	Parameters: - Manufacturing process (MA and QST) - Temperature (-40°F and 72°F) (-40°C and 22°C)	Specimens: -ASTM A706 Grade 80 and Grade 100 steel Properties to evaluate: -Yield strength -Ultimate strength -T/Y ratio -Elastic Modulus -Uniform elongation	Parameters: -Nominal strength (Grade 80 and Grade 100) -Temperature (-40°F to 72°F) (-40°C to 22°C)
	Quasi-static lateral cyclic loading column test				
2. Large-scale testing		Specimens: 6 RC Columns reinforced with ASTM A706 Grade 80 steel	Parameters: -Manufacturing process (MA and QST) -Temperature (-40°F to 72°F) (-40°C to 22°C) -Transverse steel spacing (2 inches and 4 inches)	Properties to evaluate: -Force-displacement response -Displacement capacity -Spread of plasticity -Curvature profiles	
3. Conclusion and Development of Design Recommendations					

CHAPTER 4. MATERIAL LEVEL EVALUATION

4.1. Definitions

Bending strain, ϵ_b : Strain resulting from buckling at the section of maximum curvature of a buckled bar. When a bar is subjected to compression and buckles, the bar's state of strain is a combination of axial compressive strain and bending strain from buckling, as illustrated in Figure 4-1.

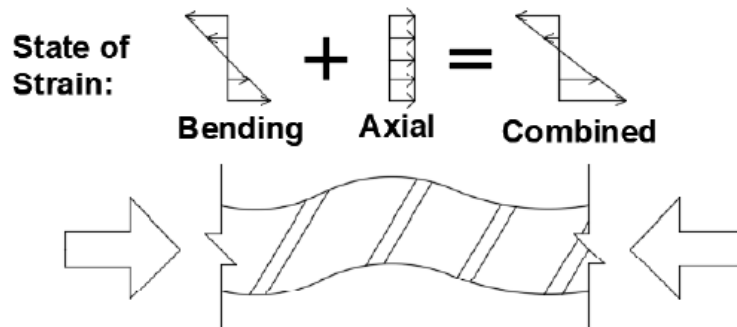


Figure 4-1 State of the strain of a buckled bar. (Barcley & Kowalsky, 2019)

Critical bending strain, ϵ_{bc} : Bending strain from buckling that leads to a brittle failure mode of reinforcing steel upon loading to tension.

Uniform Elongation, UE: Elongation at the maximum tensile force that the reinforcing steel specimen measured just before necking, fracture, or both.

Elongation at fracture, EF: Elongation measured just before the sudden decrease in force associated with fracture. This is calculated over a defined gage length that includes the fracture.

Elongation after fracture, EAF: Elongation measured by fitting the two halves of the fractured specimen together. This is calculated over a defined gage length that includes the fracture.

The EAF and the EF are not interchangeable because of differences in precision in obtaining each measurement.

Bar length, L: Clear length of the bar measured between the grips when the specimen is in place for testing.

Bar diameter, Db: Nominal diameter of the reinforcing steel specimen.

Axial displacement, Δ_{axial} : It corresponds to the total axial displacement applied in compression to achieve the desired buckling level in the specimen.

4.2. Methods

The effect of low temperatures on the mechanical properties of ASTM A706 grade 80 and grade 100 bars was assessed using BBT tests and monotonic tension tests. As previously discussed, the critical bending strain is affected by stress concentrations, which can be exacerbated by low temperatures. BBT tests were conducted to evaluate how factors that influence stress concentrations, such as the manufacturing process and bar geometry, impact the critical bending strain of HSRS at 72°F and -40°F (22°C and -40°C). Additionally, tension tests were conducted to evaluate how temperatures ranging from 72°F to -40°F (22°C to -40°C) affect the tensile properties of HSRS, including yield and strength, T/Y ratio, elastic modulus, and uniform and ultimate elongation. The experimental program followed for the material characterization of HSRS at low temperatures is presented in Table 4-1. Table 4-2 presents the evaluation goals assigned to the references stated in Table 4-1.

The experimental program for evaluating the material-level properties of HSRS at low temperatures required special considerations. These included designing and building a low-temperature testing setup, selecting the appropriate geometry to achieve testing objectives at room and low temperatures, and defining a measurement system. These considerations are presented in detail in this section, along with a description of the methods used to conduct the BBT tests and the tension test.

Table 4-1 Material-level experimental program.

Ref.	Grade	Manuf. process	Bar Surface	Test	Evaluation	Temperatures °F [°C]
1	80*	MA	Ribbed	BBT Test	Critical bending strain	72, -40 [22, -40]
2	80*	MA	Ribbed	Tension Test	Mechanical properties	72, 41, 14, 5, -4, -40 [22, 5, -10, -15, -20, -40]
3	80	MA	Smooth	BBT Test	Critical bending strain	72, -40 [22, -40]
4	80	QST	Ribbed	BBT Test	Critical bending strain	72, -40 [22, -40]
5	100	MA	Ribbed	Tension Test	Mechanical properties	72, 32, -4, -40 [22, 0, -20, -40]

* Steel designation: ASTM A706/A706M - 22a

Table 4-2 Evaluation goals of the material-level experimental program.

Evaluation goal	Refs.
Effect of the Manufacturing Process and Temperature on the Critical Bending Strain of HSRS.	1 and 4
Effect of the Bar Geometry and Temperature on the Critical Bending Strain of HSRS.	1 and 3
Effect of the Strength and Different Temperatures on Mechanical Properties of HSRS.	2 and 5

4.2.1. Experimental Setup: Liquid Nitrogen Cooling System

A liquid nitrogen (LN) spray cooling system was built and adapted to a uniaxial 110 kip (500 kN) Universal Testing Machine (UTM). The system consisted of a 230L LN Dewar, which supplied LN through a 3/8" cryogenic hose. The hose was connected to a 120V AC 1/8" solenoid valve that opened and closed to spray LN into an insulated box. A 120V PID Temperature Controller monitors the box's internal temperature through a thermocouple and

sends 'open' and 'close' signals to the solenoid valve to hold the desired temperature. The insulated box was placed on the UTM surrounding the specimen. Table 4-3 lists the materials utilized to build the cooling system, and a scheme of the system is shown in Figure 4-2. Two Type T thermocouples were used to record the temperature of the specimens during the test.

Table 4-3 Cooling system components.

Part	Function
22 PSI Liquid Nitrogen Tank (230L)	Supply LN to the system.
Cryogenic Hose 120" Length Armor 3/8"	Transfer LN from the tank to the insulated box
Cryogenic Solenoid valve 1/8" 120V AC	Control LN flows into the system.
PID Temperature Controller 120V AC	Regulate the temperature of the system.
235 PSI Safety Pressure Relief Valve 1	Protect the system from pressure buildup risk.
Nitrogen Candy Cane & T Pressure 1	Protect the system from pressure buildup risk.
Type T - Thermocouples	Measure temperature.
Foam Insulated Box	Insulate the specimen.

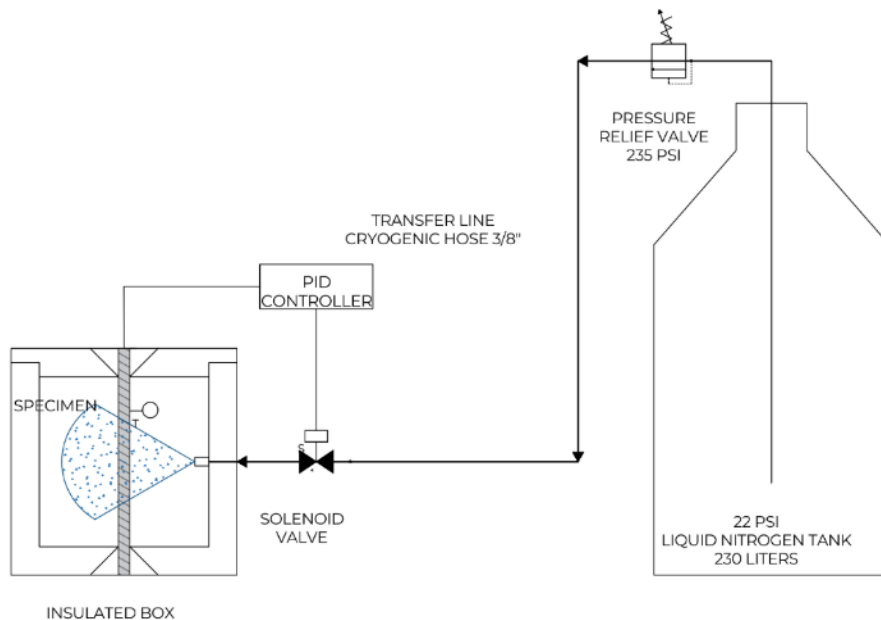


Figure 4-2 Scheme of the liquid nitrogen spray cooling system.

Two types of insulated boxes were designed to meet the specific test requirements. The insulated box built for the BBT test consisted of two separated top and bottom boxes placed on each other with an air gap of $\frac{1}{4}$ " between them (Figure 4-3). The flexible box allowed relative movement between the top and bottom portions, adapting to the specimen's vertical projection as the bar is compressed and then pulled into tension. The insulated box for the tension test setup consisted of a single insulated foam box with an "opening door" that allowed access to the specimen for placing the extensometer (Figure 4-4). The tension test box contained an 188 CFM inline fan to ensure a homogeneous temperature distribution.

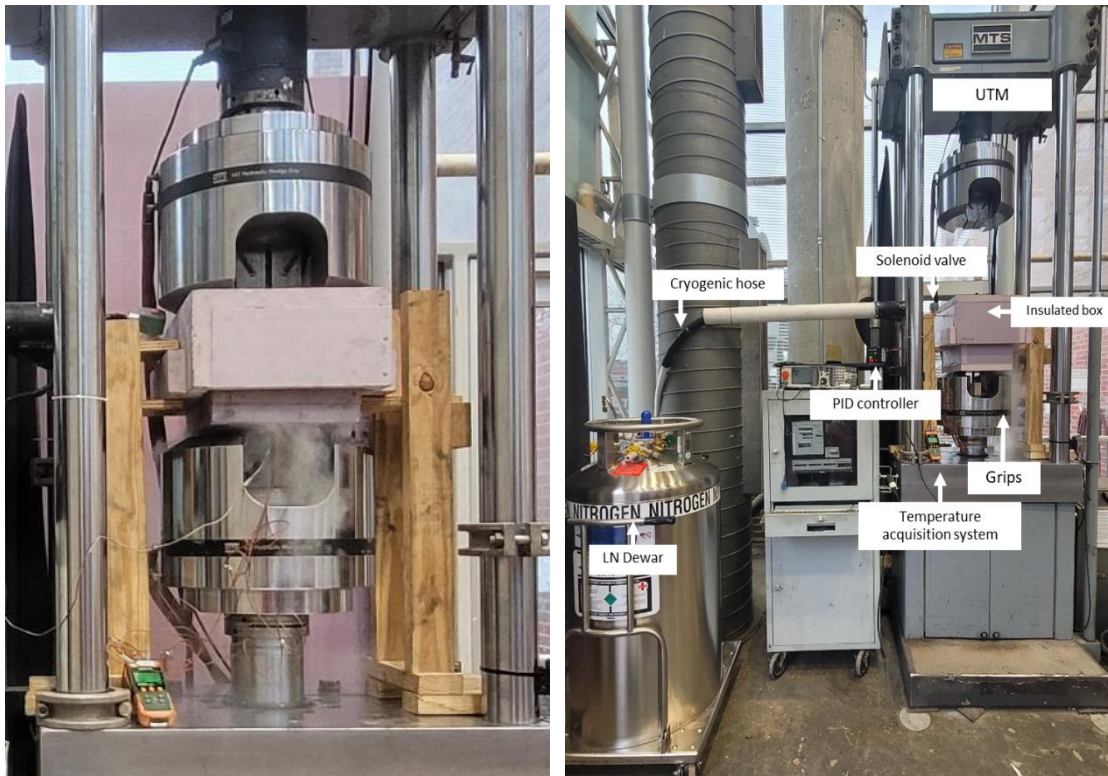


Figure 4-3 Experimental setup for low-temperature buckled bar tension test.

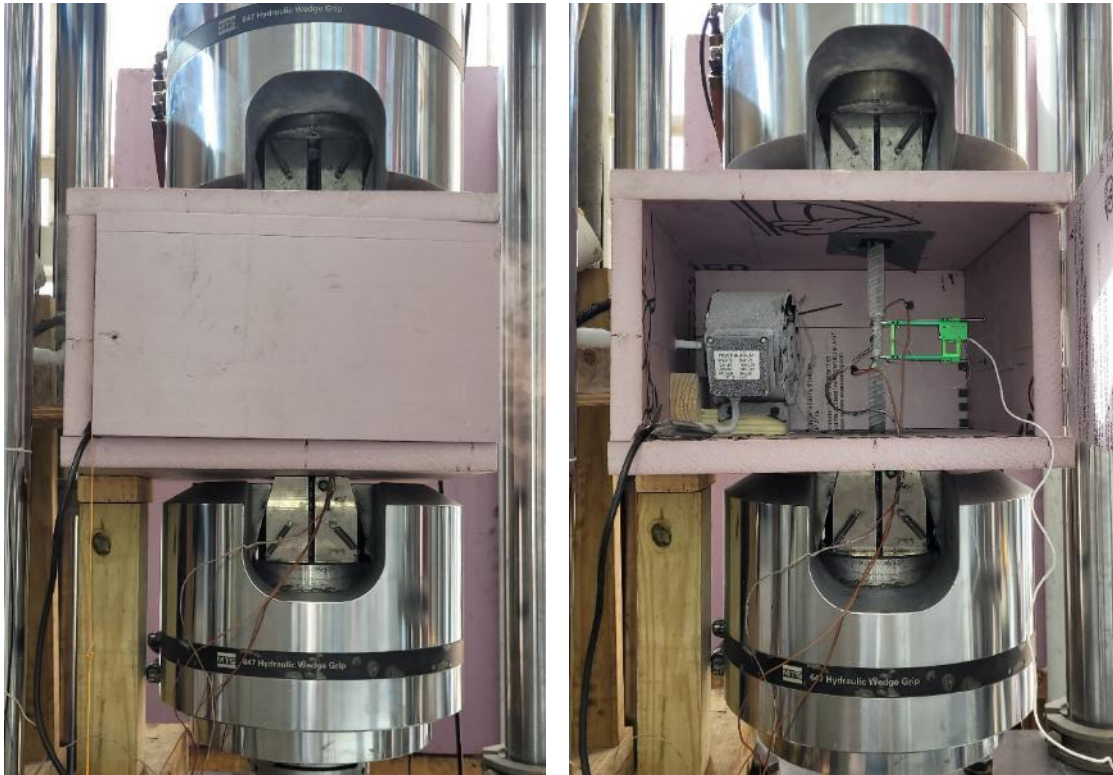


Figure 4-4 Experimental setup for low-temperature monotonic tension test.

4.2.2. Buckled Bar Tension Test

4.2.2.1. Test description

The BBT test (Barcley & Kowalsky, 2019a) is a material test that simulates the loading conditions of buckled bars in RC columns subjected to earthquakes by applying compressive and tensile loads on a reinforcing steel specimen in a UTM. The BBT test sequence consists of buckling a reinforcing bar to a predefined ε_b level and pull in tension until fracture, as shown in Figure 4-5.

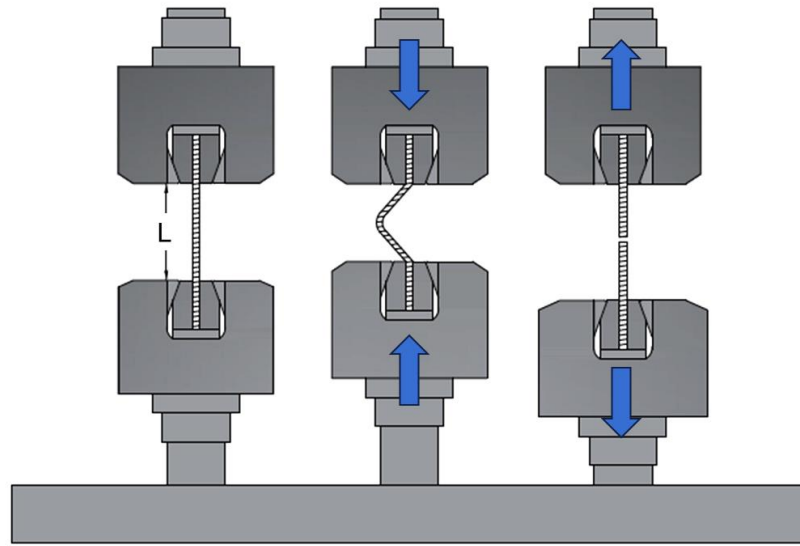


Figure 4-5 Buckled bar tension test sequence.

Two failure mode definitions are considered in the BBT test: ductile and brittle. The nature of the failure is related to measurements of bar elongation capacity, such as uniform elongation (UE), elongation at fracture (EF), or elongation after fracture (EAF), as previously defined. Ductile fractures correspond to elongation capacity measures numerically close to the ones obtained through the monotonic tension test, characterized by the presence of necking. In contrast, brittle fractures correspond to elongation capacity measures approaching zero with flat fracture surfaces.

The main objective of the BBT test is to identify a relationship between the induced bending strain and the transition of the failure mode. To do so, multiple bar samples of the same characteristics should be subjected to increasing levels of buckling-induced ε_b , controlled by a compressive axial displacement and subsequently pulled to tension until fracture, to characterize the failure mode. The critical bending strain, ε_{bc} , is obtained by a scatter plot of the ε_b , and the elongation capacity measure of each bar to identify the failure mode transition strain. The ε_{bc} corresponds to the failure mode transition range, limited by the last ductile and first brittle fractures observed within the samples in the scatter plot.

The original procedure for the BBT test requires the use of a 3D optical, non-contact measuring system, such as the Optotrak Certus HD, to obtain the strains of the specimens from the relative displacements of the sensors and to record the deformed shape of the bar during the test. However, the low-temperature setup required an alternative procedure, as the insulated box limited the vision of the sensors, making the use of the optical measurement system infeasible. Therefore, an alternative method was proposed, and it is presented later in this section as the "BBT Test Simplification." The BBT Test simplification was proposed based on hand measurements and on empirical equations developed for each set of specimens from the room-temperature results. The original and the proposed BBT test procedures are expanded next.

4.2.2.2. Original BBT test procedure for room temperature

Specimen preparation

LED caps were glued every 0.8 in (2cm) along the reinforcing steel specimen at least 24 hours before the test (Figure 4-6 top). The caps were glued using *Dow Corning 3140 RVT silicone coating*. Before placing the LED caps, the bar's surface was cleaned of oil, wax, dirt, and rust using a wire brush. LED sensors were placed directly on the LED caps at the beginning of each test to record their position throughout the test (Figure 4-6 bottom).



Figure 4-6 LED caps (top) and markers (bottom) on a reinforcing steel specimen.

Computations

Bending strain, ε_b

The ε_b is computed using the deformed shape of the bar, which is obtained by the LED sensor readings at the predefined buckling level applied to the specimen. The curvature of the section is related to the deformed shape of the bar at the neutral axis, $w(x)$, following Euler-Bernoulli beam theory Eq. (1). As the slope of the section of maximum curvature approaches zero, the expression to obtain the curvature can be simplified to Eq. (2). The ε_b is then obtained by multiplying the curvature by the bar's radius ($D_b/2$), as shown in Eq. (3).

$$\varphi = \frac{\frac{d^2w}{dx^2}}{\left[1 + \left(\frac{dw}{dx}\right)^2\right]^{3/2}} \quad \text{Equation (1)}$$

$$\varphi \approx \frac{d^2w}{dx^2} \quad \text{Equation (2)}$$

$$\varepsilon_b = \frac{\varphi D_b}{2} \quad \text{Equation (3)}$$

Elongation

The UE and EA are obtained from stress-strain results using elongations measured over an 8-gage length with the optical instrument system at peak strength and fracture, respectively, according to the definitions stated at the beginning of this document.

4.2.2.3. BBT test simplification for low temperatures

As mentioned before, the low-temperature experimental setup precluded the use of optical instrumentation because the insulated box obstructs visibility of the specimens. A simplified method was proposed to address this. The simplification involves two aspects:

first, a calibrated equation developed by polynomial fit from experimental results relating to the bar's ϵ_b to the axial displacement, Δ_{axial} , of the UTM. Secondly, marks placed at defined intervals on the reinforcing steel specimen to obtain UE and EAF according to the methods proposed in the ASTM A706M-22a and ASTM A370-20, respectively (ASTM A370-20. Standard Test Methods and Definitions for Mechanical Testing of Steel Products, 2020; ASTM International, 2022). The specimen preparation and computations to obtain ϵ_b , UE, and EAF are presented below.

Specimen preparation

The low-temperature specimens required manual marks to be placed every 0.8 in along the reinforcing steel bar to compute the UE and EAF. Although ASTM 370-20 suggests punches, this option was avoided due to concerns about the homogeneity of the punches' depths and the possibility of generating additional stress concentrations along the bar. Due to this, marks were placed using ink markers on the longitudinal rib. The specimen surface was sandblasted to remove mill scale and prevent loss of ink marks as the bar elongated during the test.

Before each test, the bar specimens were pre-cooled in a chest freezer at -4 °F (-20 °C), placed on the UTM for testing, and conditioned to the desired temperature using the LN Spray cooling system for 20 min to ensure a homogeneous temperature throughout the specimen.

Computations

Empirical equations to obtain ϵ_b at Low-temperature for BBT tests

Experiments conducted by Barclay & Kowalsky (2019) showed a correlation between the ϵ_b and the Δ_{axial} . Eq. (4) was obtained from this correlation through a second-order polynomial fit, as shown in Figure 4-8.

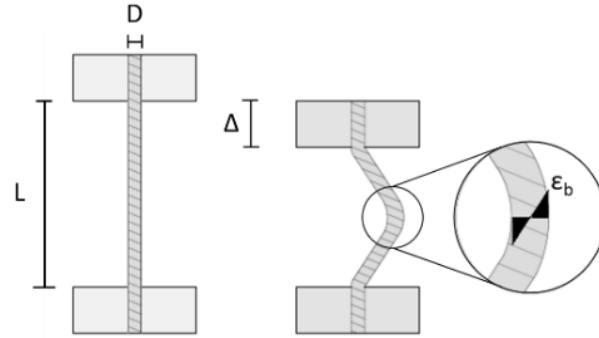


Figure 4-7 Parameters used in the BBT test simplifications.

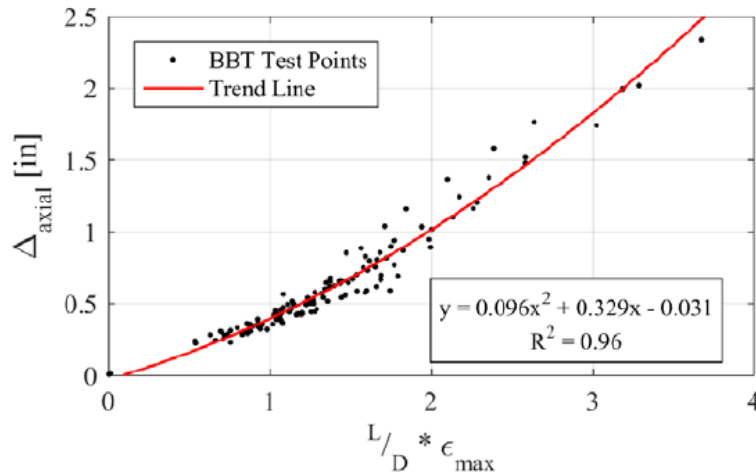


Figure 4-8 Axial displacement versus bending strain. (Barclay & Kowalsky, 2019).

$$\Delta_{axial} = 0.096 \left(\frac{L}{Db} \epsilon_b \right)^2 + 329 \left(\frac{L}{Db} \epsilon_b \right) - 0.031 \quad \text{Equation (4)}$$

However, Eq. (4) was derived from 133 experimental results from specimens with L/Db ranging from 8 to 16, with most specimens' L/Db between 8 and 11. For the ASTM A706 Ribbed MA and QST Grade 80 (references 1 and 4 in Table 4-1), 28 #6 bars with an

L/D ratio of 13.33 were tested at room temperature. The L/Db of the tested bars varied from that used to develop Eq. (4), resulting in a significant variation when obtaining ϵ_b . Because of the equation's sensitivity to the L/Db ratio, empirical equations were derived from room-temperature results.

The room temperature results on ASTM A706 ribbed MA and QST Grade 80 showed a linear correlation between the Δ_{axial} and the ϵ_b of the specimens, as presented in Figure 4-9, which were used to develop the empirical equation to process the BBT data at low temperature, as presented in Table 4-4.

Table 4-4 Empirical equation developed for low-temperature BBT tests.

Empirical equation		Specimen descriptions	Ref*	Graph
$\Delta_{axial} = 0.5451 \left(\frac{L}{D} \epsilon_b \right) - 0.2775$	Equation (5)	Ribbed MA and QST Grade 80	1,4	Figure 4-9

* Reference from Table 4-1

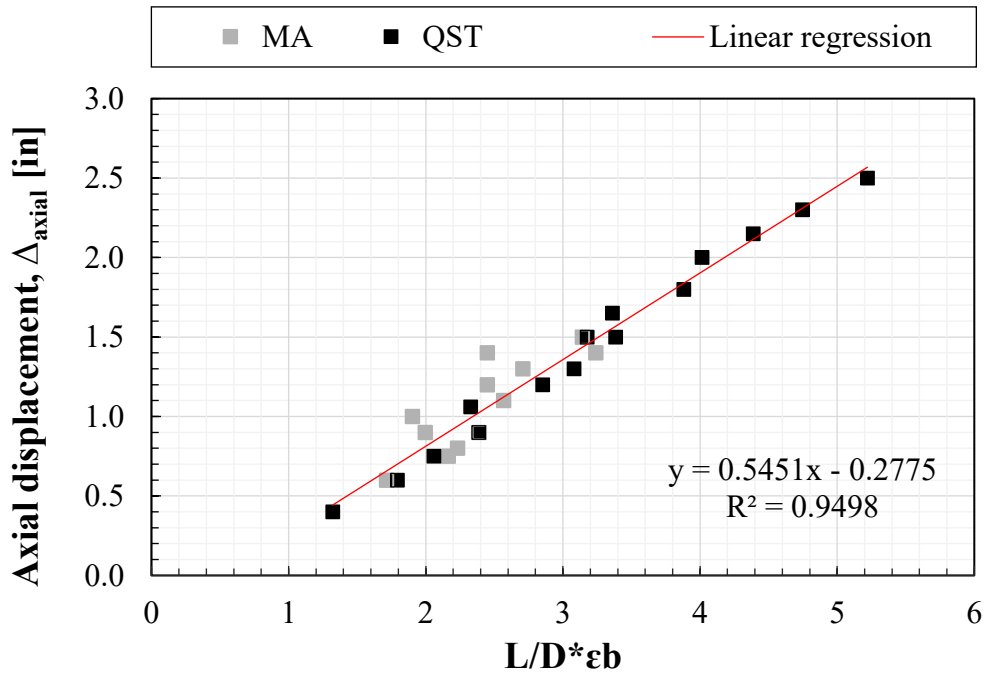


Figure 4-9 Axial displacement versus bending strain times L/D ratio for the room temperature BBT test results on Ribbed MA and QST Grade 80 (Reference from Table 4-1).

Elongation

For the simplified BBT method, the UE and EAF were obtained manually from the marks placed on the specimen after rupture. The UE was obtained following the manual method presented in ASTM A706-22a A2.2. Meanwhile, the elongation after fracture (EAF) was obtained by fitting the two broken portions of the specimen and measuring the deformation over an 8-inch gauge, as specified in ASTM A370-20 14.4.

4.2.2.4. BBT Test program

As mentioned at the beginning of this section, BBT tests were performed on ASTM 706-22a MA and QST ribbed grade 80 bars and MA smooth bars. The test matrix for the BBT tests is presented in Table 4-5. It is important to recall that the objective of the BBT test experimental program is to directly address the effect of temperature on ϵ_{bc} across different steel properties, such as manufacturing processes (MA and QST) and geometries (Ribbed and Smooth bars). Therefore, each group of specimens is evaluated at both low and room temperatures, ensuring that temperature remains the single variable.

Table 4-5 Test matrix for the BBT test program.

Ref. *	MILL	Bar Size	Fy (Ksi)	Geometry	Manufacturing process	Temperature °F (°C)
1	NUCOR	#6	80	Ribbed	MA	-40 (-40)
1	NUCOR	#6	80	Ribbed	MA	72 (22)
3	NUCOR	#6	80	Smooth	MA	-40 (-40)
3	NUCOR	#6	80	Smooth	MA	72 (22)
4	CMC	#6	80	Ribbed	QST	-40 (-40)
4	CMC	#6	80	Ribbed	QST	72 (22)

*From Table 4-1

4.2.3. Monotonic Tension Test

4.2.3.1. Test description

The monotonic tension test subjects a reinforcing steel specimen to pure tension by pulling it in a UTM until fracture occurs. The force and elongation are measured and recorded during the test to obtain the specimens' material properties. The tension test follows the specifications in the ASTM A370-20 (ASTM A370-20. Standard Test Methods and Definitions for Mechanical Testing of Steel Products, 2020) and the ISO 6892-3 (*ISO 6892-3:2015 - Metallic Materials — Tensile Testing — Part 3: Method of Test at Low Temperature*, n.d.) for test methods on tensile testing at low temperatures in Metallic Materials.

The geometry of the reinforcing steel specimen was selected in accordance with ASTM A370-20 for a 2-inch gage length, as explained later in this chapter. An Epsilon 3542 with a 2-inch gage was utilized to measure the elongation in the test. The Liquid Nitrogen Cooling System described earlier was used to control the temperature for the low-temperature testing.

The monotonic tension test involves placing the specimen into the UTM and conditioning the bar for 20 minutes to the desired temperature. The bar is clamped into the UTM once the specimen has reached a constant temperature to avoid thermal contraction-induced stress. As gripping the specimen can introduce clamping forces into the specimen, the imposed clamping force must be removed using the UTM commands to ensure a zero-force state at the beginning of each test. The test starts once the extensometer is placed on the specimen. Due to instrument limitations, the extensometer must be removed once the bar achieves the maximum force (tensile strength). Therefore, the mechanical tensile properties obtained in this study are addressed up to the peak strength of the specimens.

4.2.3.2. Specimens' preparation

Monotonic tension tests are conducted using machined-down specimens. The specimen's geometry was selected to reduce the temperature gradient along the bar and comply with the temperature control requirements presented in ISO 6892-3 (*ISO 6892-3:2015 - Metallic Materials — Tensile Testing — Part 3: Method of Test at Low Temperature*, n.d.).

The specimen preparation for the monotonic tension test includes cutting the bars to 24 in (61 cm) and then machining the reinforcing steel bar to the selected geometry at the bar's central height. Before each test, the bar specimens are pre-cooled in a chest freezer at -4 °F (-20 °C), placed on the UTM for testing, and conditioned to the desired temperature using the LN Spray cooling system for 20 min to ensure a homogeneous temperature along the specimen.

4.2.3.3. Determination of tensile parameters

The mechanical properties assessed for this study through the monotonic tension test include elastic modulus (E), yield and tensile strength (f_y and f_u), yield strain (ϵ_y), uniform elongation (UE), and T/Y ratio. Force and elongation were recorded during the test until the specimen reached maximum stress to prevent extensometer damage caused by rupture. Stress-strain curves were obtained by using the specimen's initial diameter to calculate stress, and strain measurements were obtained directly from a 2-inch gage Epsilon extensometer. The mechanical properties were obtained in accordance with the definitions in ASTM A370-20. The yield strength was determined using the 2% offset method as specified in ASTM A370-20. The modulus of elasticity was obtained from the slope of the linear portion of the stress-strain curve, and the yield strain corresponded to the yield stress divided by the elastic

modulus. The peak stress corresponded to the maximum stress applied force on the bar, and the uniform elongation corresponds to the elongation measured at the peak stress.

4.2.3.4. Geometric selection of the specimen

A machined-down specimen was proposed to overcome the temperature limitations of the experimental setup. The LN cooling system was adapted to the UTM, with the restriction that the equipment's grips should remain outside of the insulated box due to the UTM's temperature specifications. This restriction on the experimental setup generates a temperature gradient along the reinforcing steel bar, as the sampled extremes are exposed to room temperature. Control of the temperature gradient in the specimen is critical for the monotonic tension test, as temperature differences shift failures to the warmer portions, where strength is lower, thereby affecting the reliability of the test.

To minimize variability in the results, ISO 6892-3 specifies that the temperature variation along the specimen should remain lower than 5.4 °F (3 °C). Using 2-foot-long reinforcing steel bars with a reduced cross-section in the central portion enabled the temperature requirements to be met. The reduced section was the test specimen, which remained entirely contained within the insulated box, and the non-machined bar sections extended from the specimen to the grips of the UTM as “specimen extensions”. The geometry of the machined-down portion of the reinforcing steel bar was selected in accordance with ASTM A370-20 for bar specimens.

A preliminary set of tests was conducted to evaluate the impact of reducing the cross-section of MA and QST bars on different machine-down sections. The primary concern relied on the ability of the modified specimen to represent the mechanical properties of the unmodified bars, especially for the QST bars, as the higher strength of these bars is provided

by the combination of a ductile ferrite-pearlite core and the hard-tempered martensite exterior, and the machining process reduces the hard-tempered martensite exterior.

Figure 4-10 presents two geometric definitions assessed for this study (specimens type 1 and 2) selected according to ASTM A370-20 for 2-inch and 1.5-inch gage length, respectively. The results obtained with the proposed geometries were compared with those obtained using full bars (unaltered specimens). Table 4-6 shows the test matrix used to conduct this preliminary assessment. Figure 4-11 and Figure 4-12 present the comparison of the specimens for different geometries. The geometric evaluation was performed at room temperature.

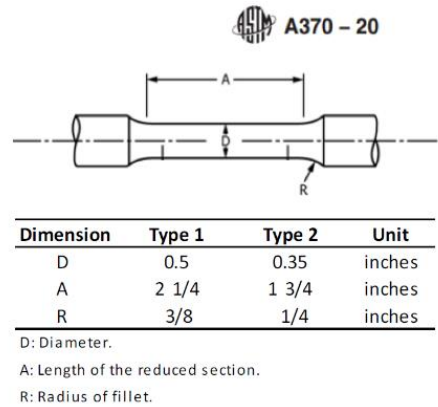


Figure 4-10 Geometric specifications for the reduced cross-section machine specimens. Adapted from ASTM A370-20.

Table 4-6 Test matrix for selection of tension test specimens.

Manufacturing process	Nominal bar size	Specimen type	Cross-sectional area (in ²)	Area reduction	Label
MA	#5	Full bar	0.307	-	MA_Full_#5
MA	#5	Type 1	0.196	36%	MA_0.5_#5
MA	#7	Full bar	0.601	-	MA_Full_#7
MA	#7	Type 1	0.196	67%	MA_0.5_#7
QST	#6	Full	0.442	-	QST_Full_#6
QST	#6	Type 1	0.196	56%	QST_0.5_#6
QST	#6	Type 2	0.096	78%	QST_0.35_#6
QST	#8	Full	0.785	-	QST_Full_#8
QST	#8	Type 1	0.196	75%	QST_0.5_#8
QST	#8	Type 2	0.096	88%	QST_0.35_#8

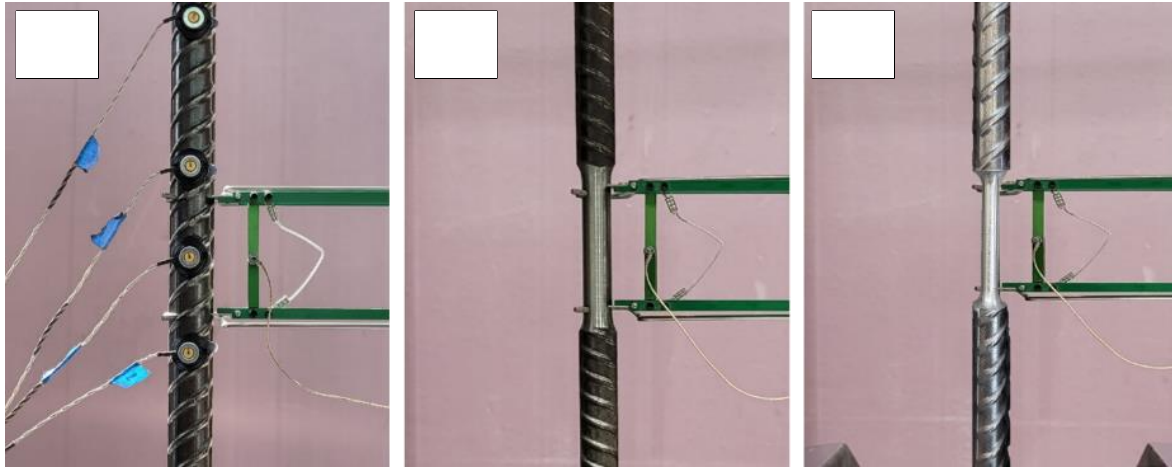


Figure 4-11 #6 QST bar specimens used in the preliminary tensile tests to define the geometry of tensile test specimens. a) Full bar. b) Type 1 specimen, and c) Type 2 specimen.

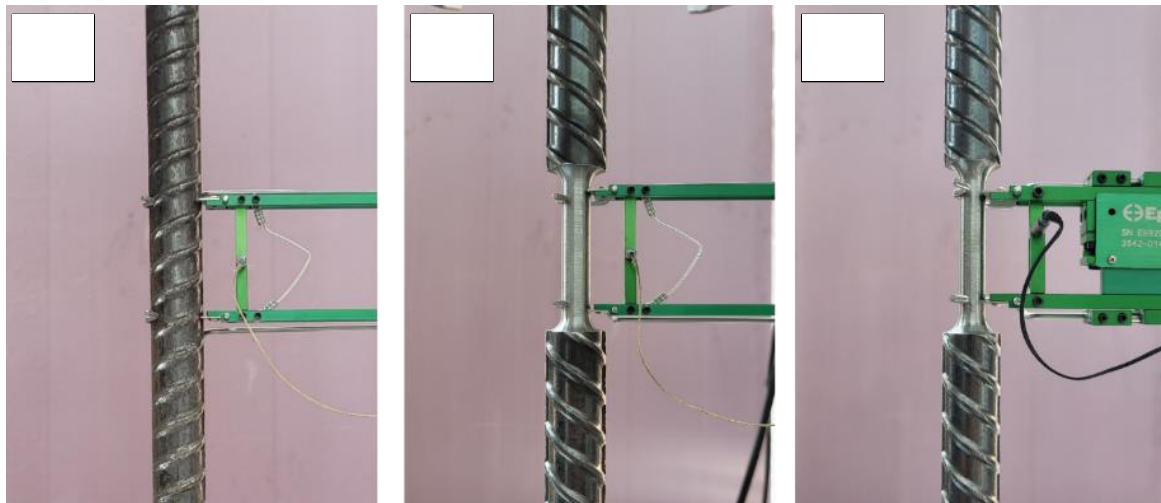


Figure 4-12 #8 QST bar specimens used in the preliminary tensile tests to define the geometry of tensile test specimens. a) Full bar. b) Type 1 specimen, and c) Type 2 specimen.

Impact of area reduction on MA specimens for tension test specimen selection.

As presented in Table 4-6, #5 and #7 bars were used to evaluate the feasibility of using machined-down specimens on MA bars. The bars were machined down to the type 1 specimen, resulting in area reductions of 36% and 67%, respectively. The machined-down specimens were compared with the results obtained from unaltered bars. Figure 4-13 shows

the strain-strain curves for the 4 types of specimens evaluated for this phase. The comparison between the mechanical properties of the machined and unaltered #5 and #7 MA specimens is shown in Table 4-7.

Results showed an increase in strength in the machined-down specimens, ranging from 7% to 13%. T/Y ratios remained mainly constant, regardless of the cross-sectional area being removed. The elastic modulus increased by 21% and 12% for the #5 and #7 specimens, respectively, and uniform elongation presented a 0% to 22% increment. The increase in the strength of the MA is attributed to centerline segregation, an uneven distribution of alloys along the steel structure that can lead to microstructural differences across the steel cross-section.

Since the strength increment was consistent across area reductions and the specimens to be evaluated within each experimental group of the program will have the same geometric properties, machined-down MA bars were defined as suitable for assessing the impact of temperature on the tensile properties of MA bars in this study.

Table 4-7 Tensile properties comparison between Type 1 and Unaltered MA specimens.

Properties	MA #5			MA #7		
	Full	Type 1	Type1 / Full	Full	Type 1	Type1 / Full
Yield strength (ksi)	85.2	95.1	12%	83.4	89.1	7%
Tensile strength (ksi)	112	126	13%	114	124	9%
T/Y	1.32	1.33	1%	1.36	1.39	2%
Elastic Modulus (ksi)	24610	29747	21%	26870	30128	12%
Uniform elongation	0.10	0.10	0%	0.09	0.11	22%

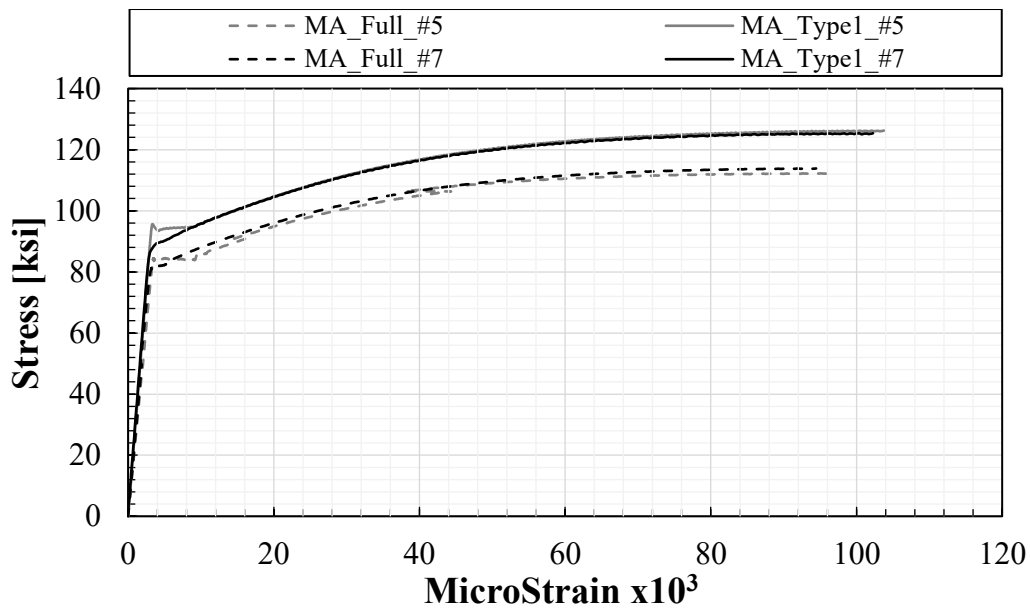


Figure 4-13 Stress-strain curve using different geometries of MA Grade 80 specimens.

Impact of area reduction on QST specimens for tension test specimen selection

Type 1 and 2 specimens were machined from #8 and #6 QST bars. The specimens were subjected to tension tests at room temperature, and the tensile properties were compared to those obtained from their respective unaltered bar tension test results. Type 1 and type 2 specimens resulted in a cross-sectional area reduction of 36% and 67% for the #6 bars and 75% and 88% for the #8 bars, respectively, as expressed in Table 4-6. The stress-strain curves for the #6 and #8 specimens are shown in Figure 4-14 and Figure 4-15. The comparison between the mechanical properties of the machined and unaltered #6 and #8 QST specimens is shown in Table 4-8 and Table 4-9 for the type 1 and type 2 specimens, respectively.

As expected, decreasing the cross-sectional area of the QST specimens resulted in a considerable change in the bars' material properties. Machining on the QST specimen led to a lower yield strength that varied with the area reduction. As a result, T/Y ratios presented significant variations ranging from 8 to 27%. Elongation capacity reached increments up to

74% for the bars with a higher percentage of area removed. The distribution in the variation of material properties in the MA and QST is presented in Figure 4-15, which compares the normalized property values obtained from all the machined specimens to the unaltered bars. The values were normalized to the average of each data set (MA and QST). From Figure 4-15, it is possible to observe a greater scatter in the properties obtained on the QST bars than on the MA bars. Due to this significant variation in the material properties of the QST bars, influenced by the amount of cross-sectional area removed, the experimental program to evaluate the tensile properties of HSRS at low temperatures was limited to MA bars using Type 1 specimens.

Table 4-8 Tensile properties comparison between Type 1 and Unaltered QST specimens.

Properties	QST #6			QST #8		
	Full	Type 1	Type1 / Full	Full	Type 1	Type1 / Full
Yield strength (ksi)	78.4	72.3	8%	80.5	69.5	14%
Tensile strength (ksi)	102	102	0%	105	104	1%
T/Y	1.3	1.41	8%	1.31	1.5	15%
Elastic Modulus (ksi)	25752	28052	9%	24847	26532	7%
Uniform elongation	0.1	0.11	10%	0.09	0.12	33%

Table 4-9 Tensile properties comparison between Type 2 and Unaltered QST specimens.

Properties	QST #6			QST #8		
	Full	Type 2	Type2 / Full	Full	Type 2	Type2 / Full
Yield strength (ksi)	78.4	70.9	10%	80.5	61.9	23%
Tensile strength (ksi)	102	101	1%	105	103	2%
T/Y	1.3	1.42	9%	1.31	1.67	27%
Elastic Modulus (ksi)	25752	21192	18%	24847	20302	18%
Uniform elongation	0.1	0.15	50%	0.09	0.16	78%

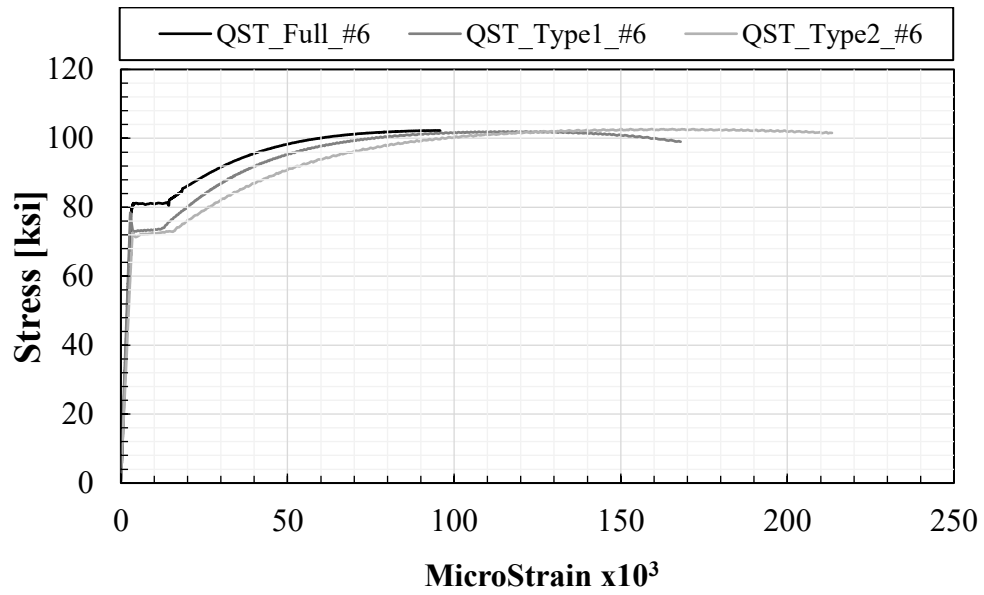


Figure 4-14 Stress-strain curve using different geometries of QST Grade 80 #6 specimens.

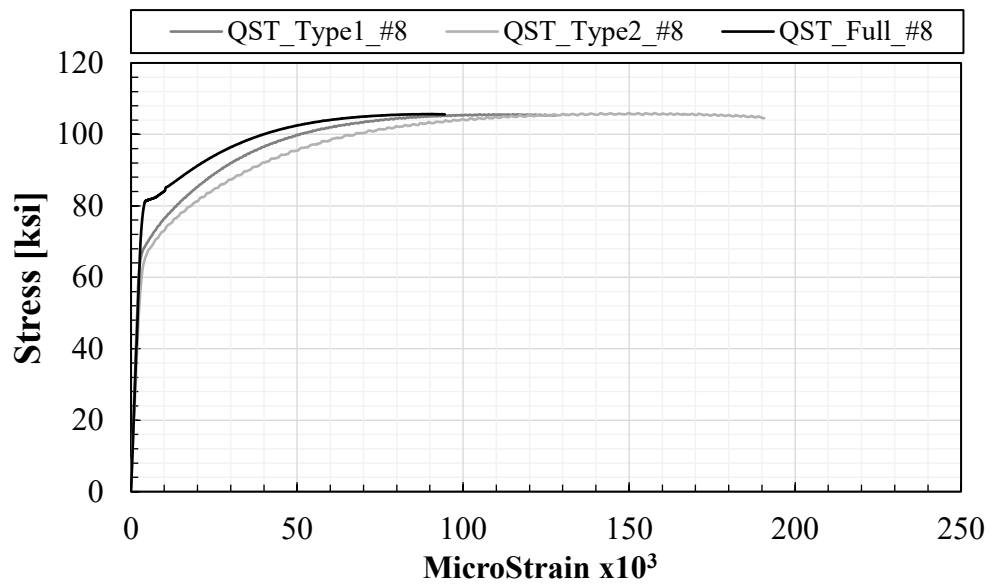


Figure 4-15 Stress-strain curve using different geometries of QST Grade 80 #8 specimens.

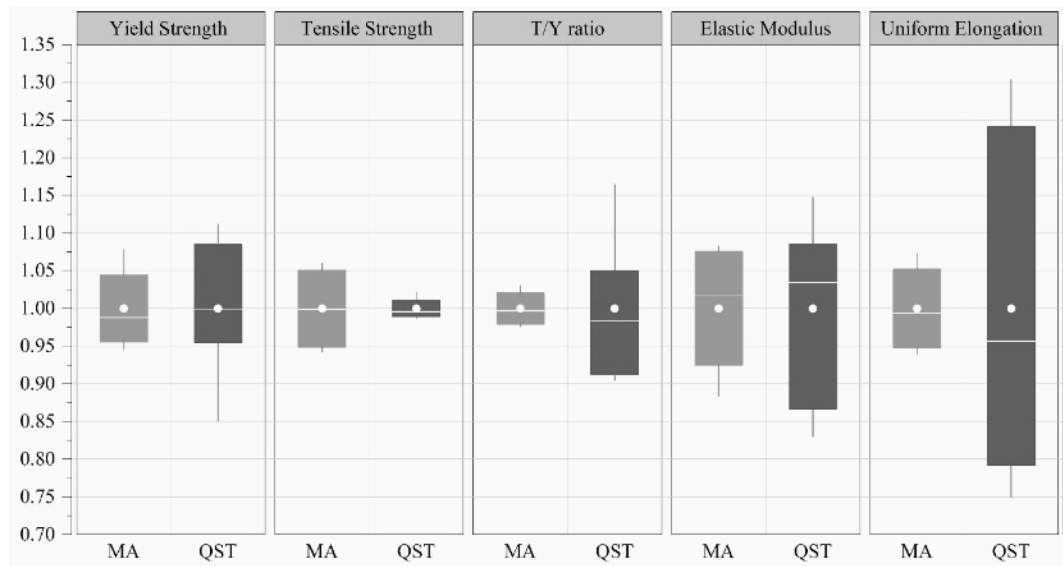


Figure 4-16 Distribution of the tensile properties obtained from MA and QST specimens machined to different cross-sectional areas. All values are normalized to the mean for each dataset.

4.2.3.5. Testing program

Monotonic tension tests were conducted at temperatures ranging from 72°F to -40°F (22°C to -40°C) to evaluate the temperature effect on the tensile properties of MA ASTM 706 gr. 80 and gr. 100 bars, using the geometry of the type 1 specimen specified previously. The test matrix is presented in Table 4-10.

Table 4-10 Tension test matrix.

Ref.*	MILL	Manufacturing process	Bar Size	Diameter type 1 specimen (in)	fy (ksi)	No. of specimens	Temperature °F (°C)
2	NUCOR	MA	#7	0.5	80	3	-40 (-40)
2	NUCOR	MA	#7	0.5	80	3	-4 (-20)
2	NUCOR	MA	#7	0.5	80	3	5 (-15)
2	NUCOR	MA	#7	0.5	80	3	14 (-10)
2	NUCOR	MA	#7	0.5	80	3	41 (5)
2	NUCOR	MA	#7	0.5	80	3	72 (22)
5	NUCOR	MA	#6	0.5	100	2	-40 (-40)
5	NUCOR	MA	#6	0.5	100	2	-4 (-20)
5	NUCOR	MA	#6	0.5	100	2	32 (0)
5	NUCOR	MA	#6	0.5	100	2	72 (22)

*From Table 4-1

In addition, a preliminary assessment of the effect of conditioning time on the material properties of the low-temperature tested specimens was conducted. The tensile properties of 5 specimens, pre-cooled for 1 hour, 24 hours, and 7 days at 5°F (-15 °C), were obtained. Results were compared with the room-temperature material test results (Table 4-11). From the results, obtaining a clear correlation between the conditioning time and the tensile properties was not possible. In general, it was observed that the relationship between the yield strength, the ultimate strength, and the T/Y obtained at different conditioning times at room temperature remained mainly constant. The Elastic Modulus was unaltered during the 24-hour temperature conditioning but increased by 10% for the 1-hour and 7-day specimens, and the uniform elongation varied from 7 to 20%. The variability in uniform elongation has been reported multiple times in the literature, with no correlation with temperature. More tests should be conducted to determine how conditioning time affects the elastic modulus. Figure 4-17 compares the average results obtained per conditioning time to those obtained at room temperature.

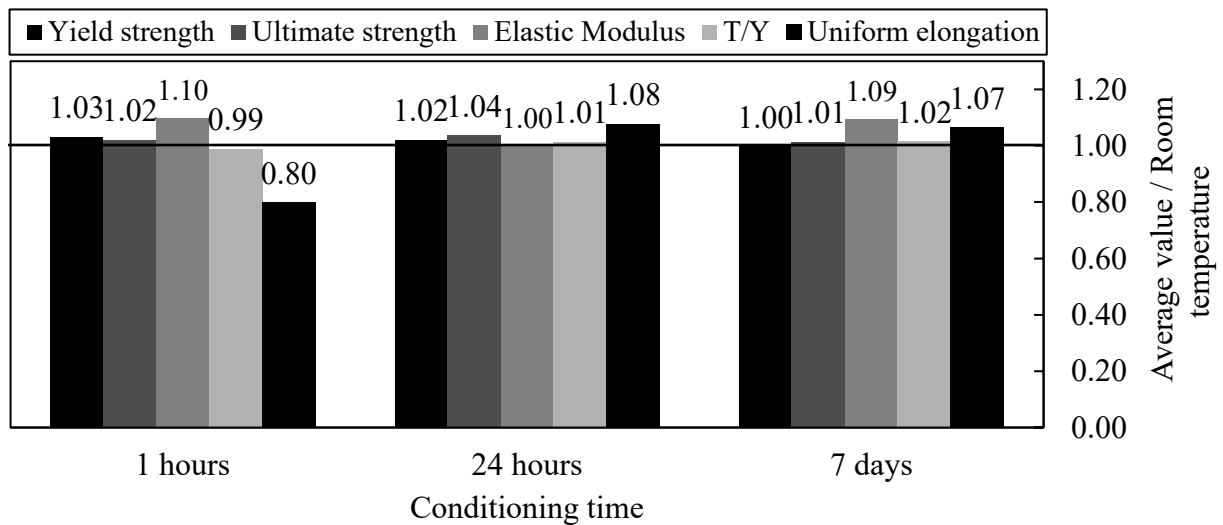


Figure 4-17 Preliminary evaluation of conditioning cooling time on the tensile properties of HSRS

Table 4-11 Results of the preliminary evaluation of using different cooling times.

Property / Cond. time	Room Temp	1 hour	24 hours	7 days
Temp.°F (°C)	5 (-15)	5 (-15)	5 (-15)	5 (-15)
Specimens	2	1	2	2
Yield strength, ksi (MPa)				
Specimen 1	89.1 (614)	91.2 (629)	89.6 (618)	89.8 (619)
Specimen 2	87.9 (606)		91.2 (629)	86.8 (598)
Average	88.5 (610)	91.2 (629)	90.4 (623)	88.3 (609)
Ultimate strength, ksi (MPa)				
Specimen 1	124 (858)	127 (874)	127 (874)	128 (881)
Specimen 2	124 (856)		131 (902)	124 (855)
Average	124 (857)	127 (874)	129 (889)	126 (868)
Elastic Modulus, ksi (MPa)				
Specimen 1	29224 (201492)	32009 (220694)	28077 (193584)	31452 (216854)
Specimen 2	29008 (200003)		30120 (207670)	32212 (222094)
Average	29116 (200748)	32009 (220694)	29099 (200627)	31832 (219474)
T/Y				
Specimen 1	1.40	1.39	1.42	1.42
Specimen 2	1.41		1.44	1.43
Average	1.40	1.39	1.43	1.43
Uniform elongation				
Specimen 1	9.36%	7.33%	9.62%	9.58%
Specimen 2	9.01%		10.1%	9.98%
Average	9.18%	7.33%	9.88%	9.78%

4.3. Results

4.3.1. Buckled Bar Tension Test

This section presents the results obtained from BBT Tests on ribbed ASTM A706 grade 80 QST and MA bars and MA smooth bars, corresponding to references 1, 3, and 4

from Table 4-1. These tests were conducted using available steel from previous research on grade 80 steel conducted by Thangjitham (2023). Based on the results, the combinations of temperature effects with manufacturing and geometric impacts on the bending strain capacity of HSRS were evaluated. Scatter plots of the applied bending strain vs. the elongation after fractures were generated to identify the transition in the failure mode corresponding to the ϵ_{bc} in all the bar groups evaluated. Figure 4-18 and Figure 4-19 present examples of the ductile and brittle failures obtained at room and low temperatures.

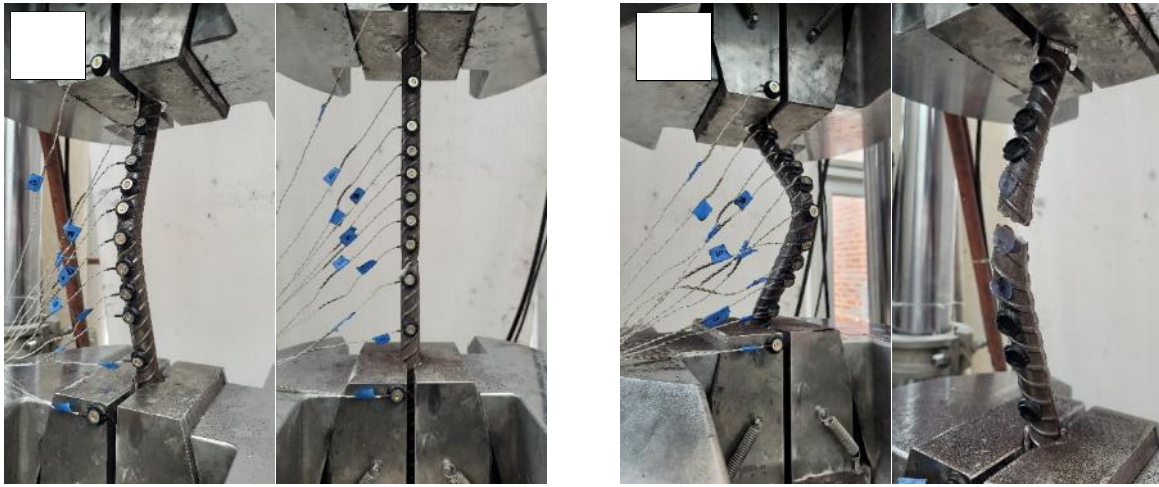


Figure 4-18 Examples of ductile(a) and brittle (b) failures obtained at room temperature BBT Test.

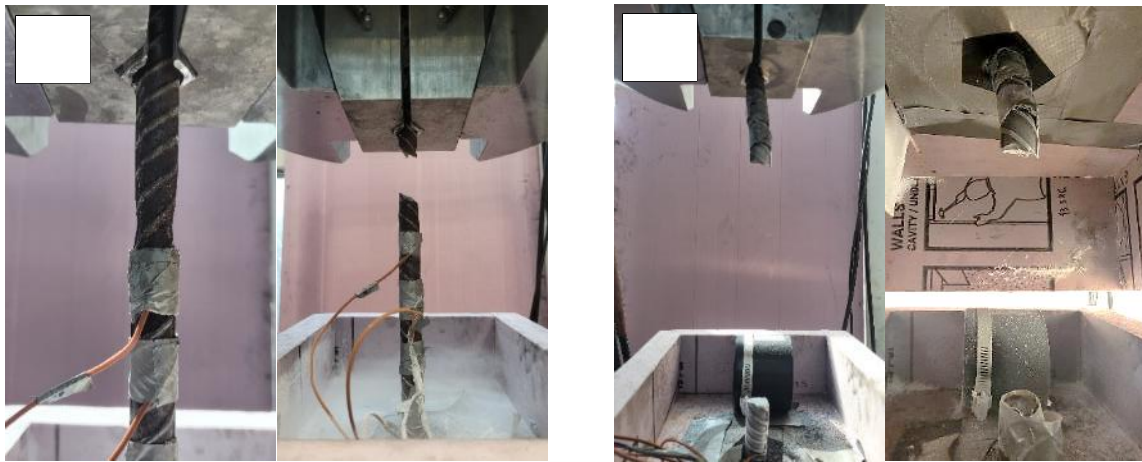


Figure 4-19 Examples of ductile (a) and brittle (b) failures obtained at low-temperature BBT Test.

4.3.1.1. Manufacturing process evaluation

The effect of the manufacturing process was evaluated by comparing the room-temperature and low-temperature BBT test results for MA and QST bars. Figure 4-20 shows a scatter of the bending strain (ϵ_b) vs. the elongation after fracture (EAF) of the QST bars. The transition in the failure mode of the QST specimens tested at room and low temperatures can be seen on the broken specimens, presented in Figure 4-21. Results from the BBT test on QST steel show similar behavior between specimens tested at room temperature and those tested at low temperatures, where the transition in failure mode from ductile to brittle occurs progressively. The only apparent difference between the results obtained at room and low temperatures is that the EAF of the low-temperature specimens tested for low bending strains ranged between 8 and 10%, lower than the EAFs at room temperature, which are nearly 15%. The temperature gradient presented during the test could explain the reduction in EAF measurement on the cold specimens. Measurements of elongation at low temperature were used to understand the transition in the failure mode. However, these values do not represent the actual elongation capacity of HSRS at low temperatures. The elongation capacity of HSRS is assessed using the tension test, which will be expanded later.

On the other hand, Figure 4-22 shows the results for the MA bars from the BBT test, illustrating a clear temperature effect on the ϵ_{bc} of the bars. MA bars typically exhibit a well-defined transition in failure mode from ductile to brittle at room temperature (Manhard & Kowalsky, 2019; Thangjitham, 2023), contrary to the QST bars. This defined transition in the failure mode is maintained at low temperatures. The temperature effect is evident when the failure mode transition occurred first in the low-temperature specimens, compared to those tested at room temperature, with ϵ_{bc} ranging from 0.14 to 0.16 at low temperature and from 0.17 to 0.19 at room temperature.

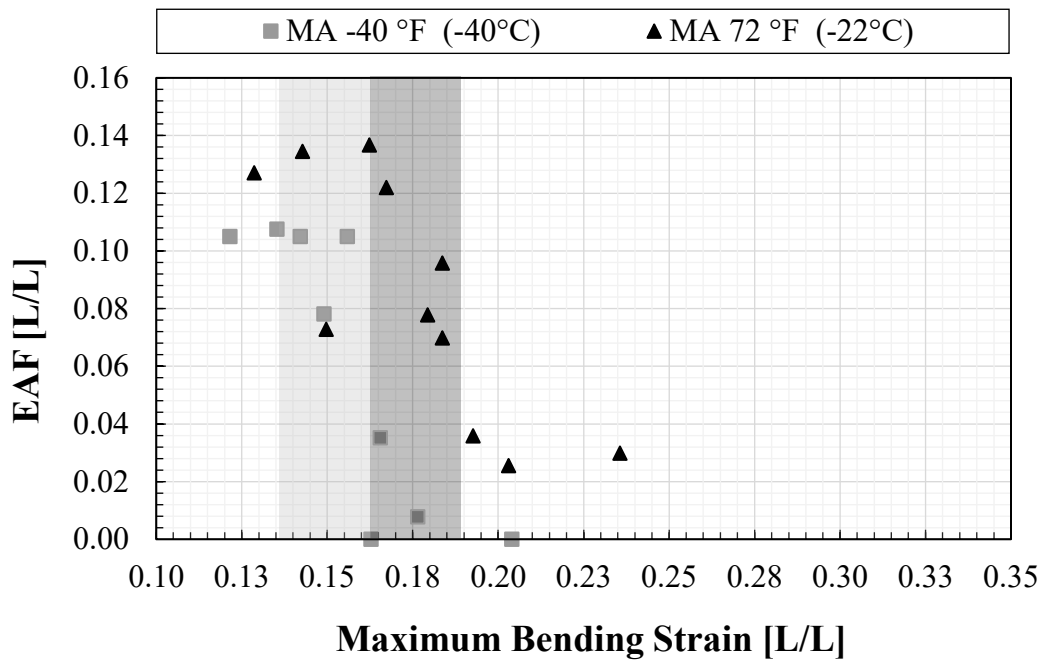


Figure 4-22 BBT Test results of ASTM A706 grade 80 MA bars.

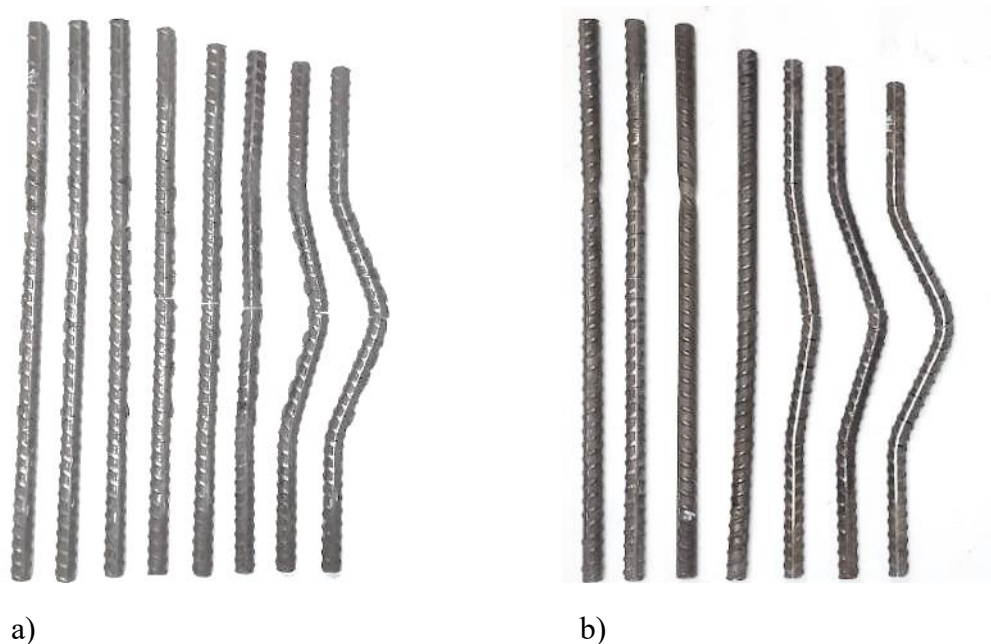


Figure 4-23 MA broken specimens, tested at Room (a) and Low (b) temperatures. Specimens were arranged from ductile to brittle failure modes (increased level of buckling)

4.3.1.2. Geometric effect evaluation (transverse deformations)

The geometric effect (the impact of rebar deformations) was evaluated by comparing the room-temperature and low-temperature BBT test results for ribbed MA and smooth MA bars. Figure 4-24 shows the bending strain vs. the EAF for the smooth MA bars. Smooth bars presented significantly higher ϵ_{bc} than ribbed bars, which is attributed to the lack of stress concentration due to the absence of ribs. Results showed that the ϵ_{bc} ranged from 0.40 to 0.44 for smooth bars at room temperature and 0.17 to 0.19 for ribbed bars. Smooth bars presented a higher reduction in the ϵ_{bc} when tested at -40°F (-40°C) compared to the ribbed bars (30% vs. 20%). However, the experimental setup may have affected the BBT test results for the smooth bars at low temperatures.

The clamping of the smooth bars on the UTM imposed deformations at the extremes of the bars. The temperature gradient on the bars exacerbated this, as the grips could not be exposed to the target temperature of -40°F (-40°C), leaving the bar ends exposed to room temperature. The lack of ribs prevented stress concentrations at the center of the bar, and the

temperature gradient shifted the location of the bar fractures to the UTM grips, as seen in Figure 4-25 (b). Examples of the ductile and brittle failures of the smooth bars at room and low temperatures are shown in Figure 4-25 and Figure 4-26.

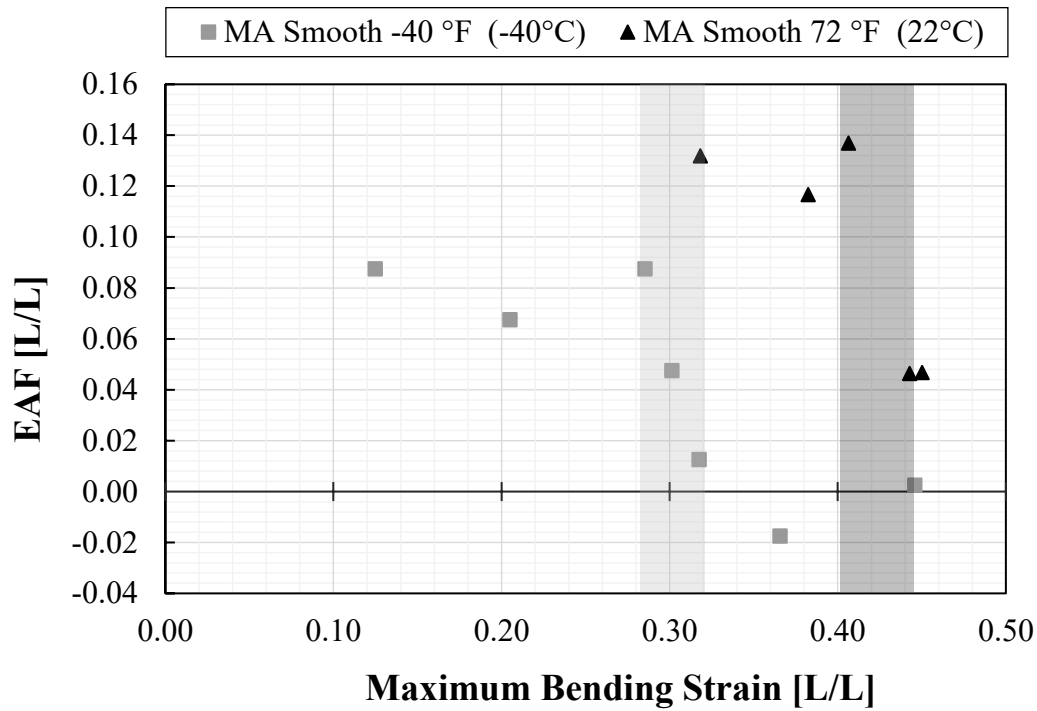


Figure 4-24 BBT Test results of smooth ASTM A706 grade 80 MA bars



Figure 4-25 MA Smooth broken specimens with ductile failures, tested at room (a) and low (b) temperatures.



Figure 4-26 MA Smooth broken specimens with brittle failures, tested at room (a) and low (b) temperatures.

4.3.2. Tension test results

Lastly, tension tests on ASTM A706 grade 80 and grade 100 steel were conducted at temperatures ranging from 72 °F to -40 °F (22°C to -40 °C). Figure 4-27 presents the average stress-strain curve obtained for the MA grade 80 bars, and Figure 4-28 the stress-strain curves obtained for all the MA grade 100 specimens. Figure 4-29 and Figure 4-30 present the temperature effect on the yield and ultimate strength of grades 80 and 100, normalized to the room-temperature results. The impact of the temperature on the modulus of elasticity, the uniform elongation, and the T/Y ratio is shown in Figure 4-31 to Figure 4-33.

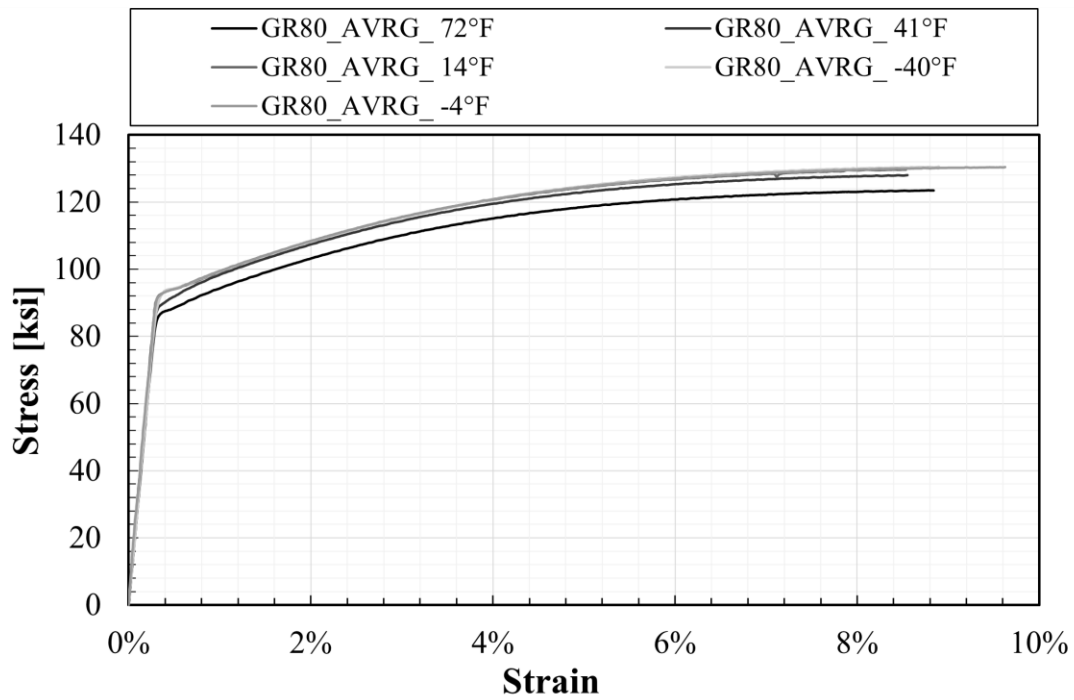


Figure 4-27 Average stress-strain curves for ASTM A706 grade 80 steel at temperatures ranged from 72°F to -40°F (22°C to -40°C)

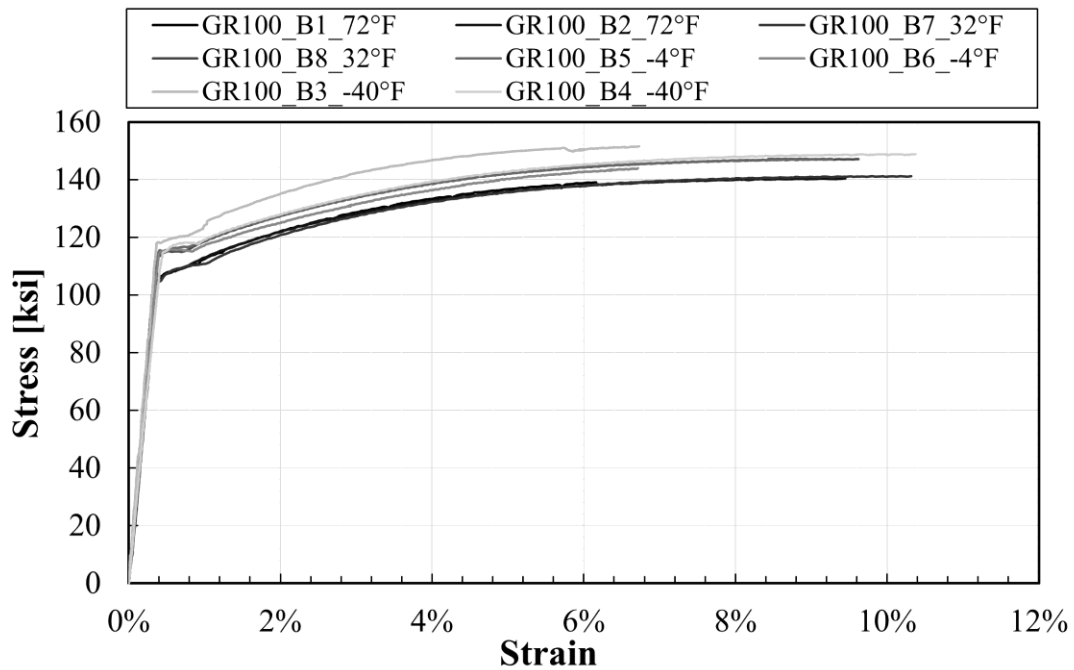


Figure 4-28 Stress-strain curves for ASTM A706 grade 100 steel at temperatures ranged from 72°F to -40°F (22°C to -40°C)

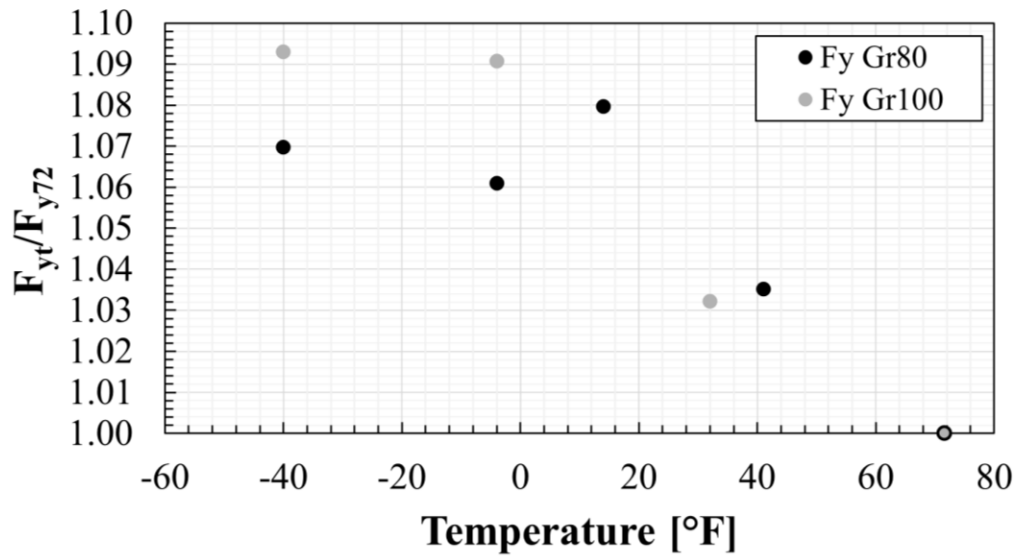


Figure 4-29 Yield strength variation as a function of temperature for ASTM A706 Grade 80 and Grade 100 reinforcing steel.

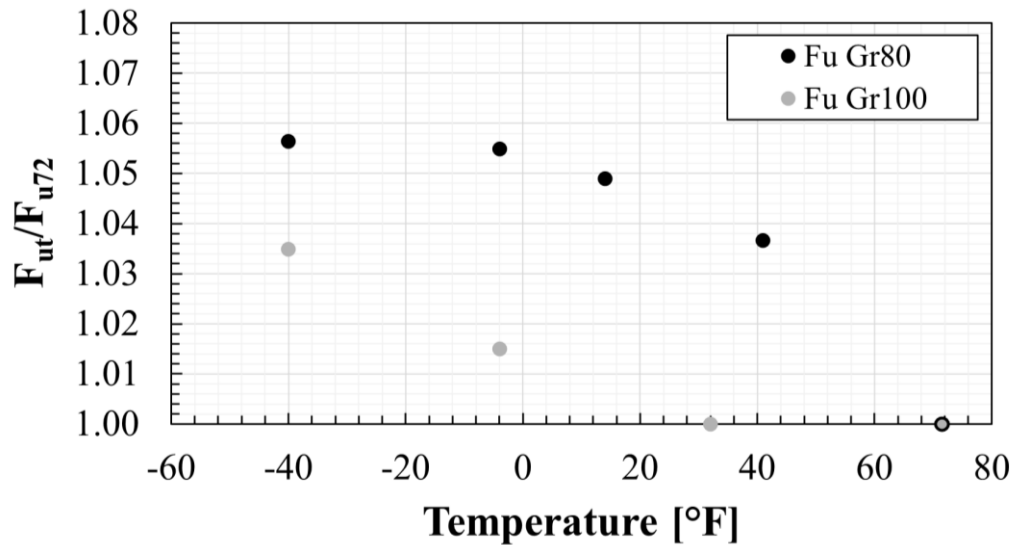


Figure 4-30 Ultimate strength variation as a function of temperature for ASTM A706 gr.80 and gr.100 reinforcing steel.

For both Grade 80 and Grade 100 steels, the yield and ultimate strength increase as the testing temperature decreases. The strength was observed to change gradually with temperature (Figure 4-29 and Figure 4-30). This trend aligns with the observations from Levings & Sritharan, (2012) on ASTM A706 grade 60 steel.

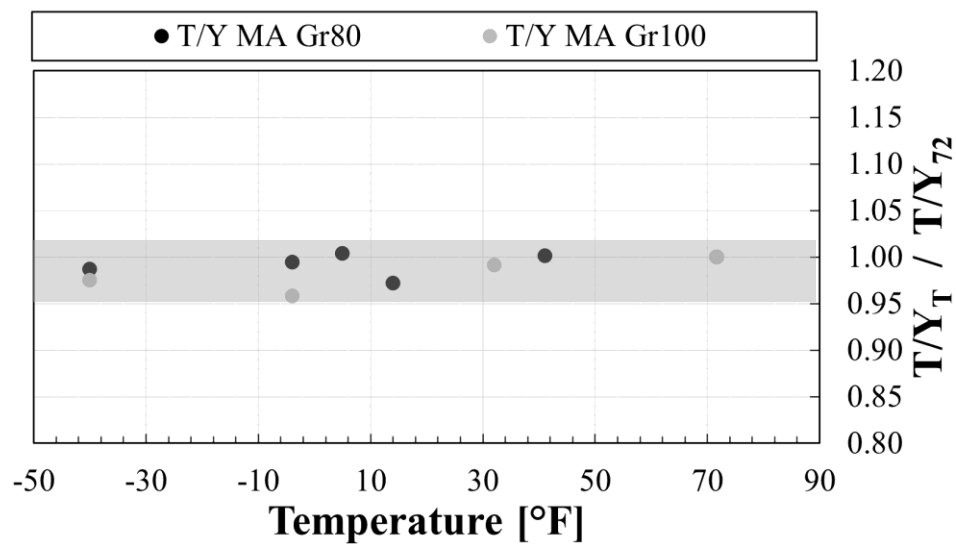


Figure 4-31 T/Y ratio variation as a function of the temperatures for ASTM A706 grade 80 and grade 100 reinforcing steel.

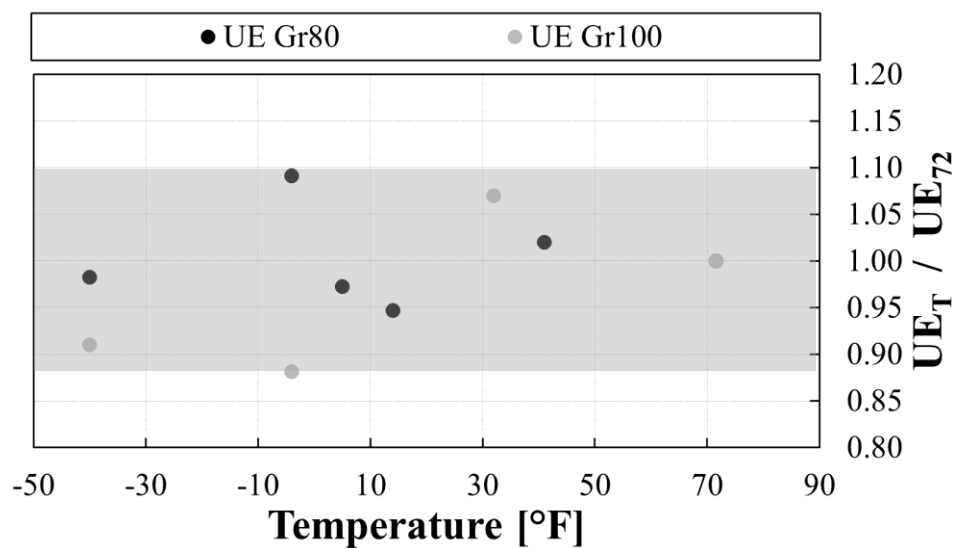


Figure 4-32 Uniform Elongation variation as a function of temperature for ASTM A706 grade 80 and grade 100 reinforcing steel.

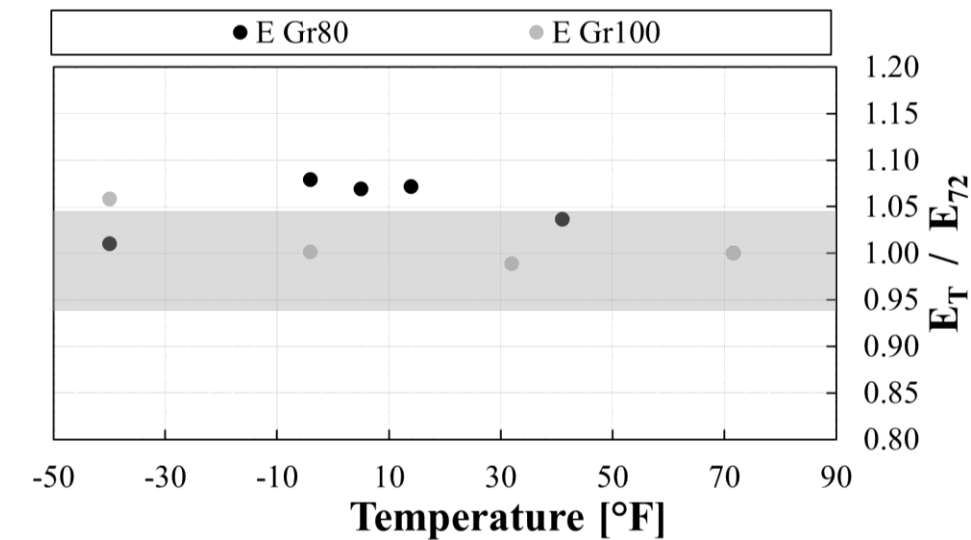


Figure 4-33 Elastic modulus variation as a function of temperature for ASTM A706 grade 80 and grade 100 reinforcing steel.

From Figure 4-31 to Figure 4-33, it was observed that the T/Y ratio remained essentially constant for grade 80 and grade 100 steels, varying from 0 to 5%. No clear trend in the elastic modulus was observed, with results at different temperatures remaining constant or increasing by up to 10% relative to room temperature. Uniform elongation showed the largest scatter in the results, with values increasing and decreasing by 10% -15% relative to room temperature, with no clear trend.

4.1. Discussion

4.1.1. Summary of BBT Test Results

The temperature effect on the ϵ_{bc} obtained from the deformed QST and MA bars and the smooth MA bars is presented in Table 4-12. These results indicate that the critical bending strain for QST bars is almost unaffected by temperature. For MA bars, a reduction of 20% in the ϵ_{bc} as the temperature decreased to -40 °F (-40 °C) was observed. In addition, smooth bars exhibited higher ϵ_{bc} at both low and room temperatures than the ribbed QST and MA bars, despite the low-temperature results showing a 30% ϵ_{bc} decrement relative to

the room-temperature results. The greater decrease in the ϵ_{bc} of the smooth bars at low temperature could be attributed to a shift in the fracture location, which altered the BBT test results.

Table 4-12 Critical bending strain on ASTM 706-22 Grade 80 MA, and QST ribbed bars, and MA smooth bars

Specimen Description	ϵ_{bc} (72°F)	ϵ_{bc} (-40°F)	ϵ_{bc} decrement (%)
QST	0.15 – 0.30	0.15 – 0.30	0
MA	0.17 – 0.19	0.14 – 0.16	20
MA Smooth	0.40 – 0.44	0.28 – 0.32	30

The material-level evaluation of the bending strain capacity of reinforced steel suggests that the manufacturing process affects the reinforcing steel's ability to sustain buckling at low temperatures, with an evident reduction in the critical bending strain of MA bars. This observation needs further investigation through large-scale testing and will be presented with the results.

4.1.2. Summary of Tension Test Results

In accordance with the trends presented in the literature review, the reinforcing steel exhibited increases in yield and ultimate stress as temperature decreased. The T/Y ratio and the elastic modulus were mainly unaffected by temperature, and the uniform elongation presented no clear trend.

Figure 4-34 and Figure 4-35 presented the increase in the yield and ultimate strength, respectively, compared to the models proposed by Montejo et al.(2008) and Levings & Sritharan (2012). For both grade 80 and grade 100 steel, temperature increases yield and ultimate strength progressively, with strength increments observed above freezing. The model proposed by Montejo et al. (2008) ignores the effect of temperatures above freezing and overpredicts the strength increase for grade 80 steel at temperatures below -4°F (-20°C).

It is shown that the effect of the low temperature on the grade 80 steel lies between the models proposed by Montejo et al. (2008) and Levings & Sritharan (2012). To represent the yield strength of grade 80 steel, an approximate 7% strength increase applies from -4°F (-20 °C) to -40°F (-40°C), and a 5% increase in strength applies to estimate the ultimate strength.

The increase in the yield strength of the grade 100 bars showed better alignment with the model proposed by Montejo et al. (2008). However, this is not maintained for the increase in the ultimate strength, which aligned better with the model proposed by and Levings & Sritharan (2012).

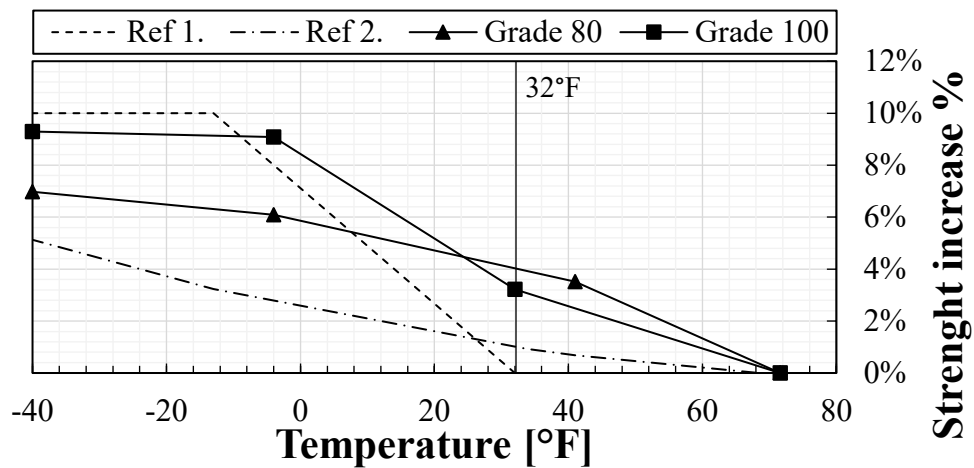


Figure 4-34 Increase in the yield strength (Fy) at various temperatures.
Ref1: Montejo et al., (2008) , and Ref2: Levings & Sritharan, (2012)

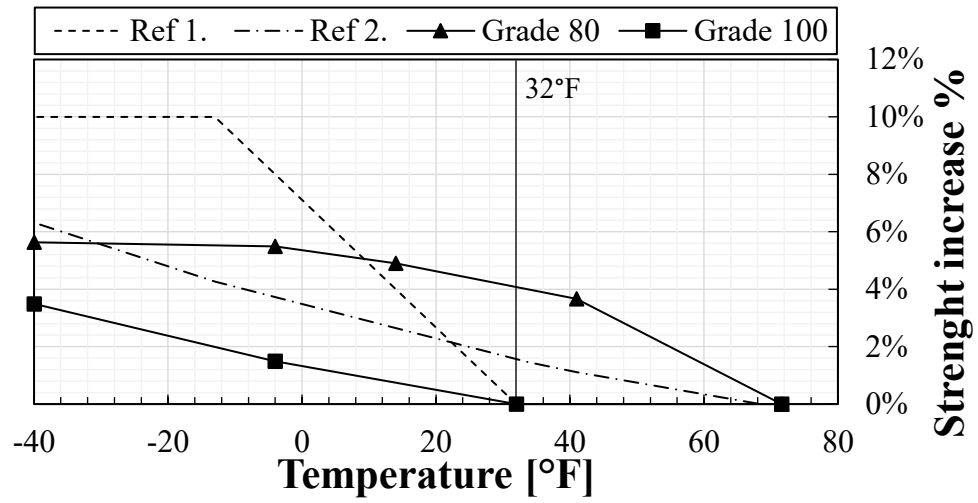


Figure 4-35 Increase in the ultimate strength (Fu) at various temperatures.
 Ref1: Montejo et al., (2008) , and Ref2: Levings & Sritharan, (2012)

CHAPTER 5. MEMBER LEVEL EVALUATION

This section presents the large-scale evaluation of six RC bridge column specimens reinforced with MA and QST grade 80 steel, subjected to quasistatic lateral loading at temperatures from 72°F to -40°F (22°C to -40°C). The design and detailing of the RC column specimens are initially presented, followed by the construction process, test definitions, experimental setup, instrumentation, and experimental observations. The chapter concludes with a comparison of the seismic behavior of the specimens at room and low temperatures, along with main observations on the low-temperature effect for both manufacturing methods (MA and QST).

All columns were loaded under the same displacement-based loading protocol to ensure uniform input and facilitate direct comparison of member-level responses. The RC columns were instrumented using strain gages, linear potentiometers, string potentiometers, inclinometers, and thermocouples. These instruments were used to measure global and local responses, including force-deformation hysteretic behavior, strain profiles along the longitudinal steel, curvature distributions, and temperature variations throughout the tests. A constant axial load of 123 kips was applied to the columns to simulate the vertical loads of a bridge column for a designed ALR of 8%

5.1. Experimental program

The large-scale experimental program includes six RC column specimens reinforced with ASTM A706 grade 80 MA and QST steel. The first four specimens share the same detailing, with the only difference being the manufacturing process for the longitudinal reinforcing steel. Each pair of columns (reinforced with MA or QST longitudinal steel) was evaluated at room and low temperatures to assess the manufacturing impact on the low-temperature seismic behavior of RC columns and to compare directly with the room-temperature references. Following the first four columns, a second pair of columns was tested with a larger transverse steel spacing for both manufacturing processes at low temperature. The larger spacing was used to aggravate buckling conditions and to observe the main differences in the seismic behavior at low temperatures between the two manufacturing processes. The test matrix for the six specimens is presented in Table 5-1. The row labeled "Column specifications" in Table 5-1 presents the properties of each test, following the notation: Manufacturing process (MA/QST) – Test temperature (RT/LT) – spiral separation (2in/4in).

Table 5-1 Large-scale testing test matrix

Col #	Column specifications	Diam (in)	Length (in)	L/D ratio	Longitudinal Steel	Transverse Steel	ALR	Temp. °F (°C)
Col 1	MA-RT-2in	18	65	3.61	10 #6 (1.74%)	#3 @ 2" (1.23%)	8%	72 (22)
Col 2	QST-RT-2in	18	65	3.61	10 #6 (1.74%)	#3 @ 2" (1.23%)	8%	72 (22)
Col 3	MA-LT-2in	18	65	3.61	11 #6 (1.74%)	#3 @ 2" (1.23%)	8%	-40 (-40)
Col 4	QST-LT-2in	18	65	3.61	12 #6 (1.74%)	#3 @ 2" (1.23%)	8%	-40 (-40)
Col 5	MA-LT-4in	18	65	3.61	13 #6 (1.74%)	#3 @ 4" (0.61%)	8%	-40 (-40)
Col 6	QST-LT-4in	18	65	3.61	10 #6 (1.74%)	#3 @ 4" (0.61%)	8%	-40 (-40)

5.2. RC column specimens

5.2.1. RC columns design

The specimens were designed following a performance-based design philosophy, in which the expected material properties (Table 5-2) were used to define detailing capable of achieving ductile failure, characterized by bar buckling followed by bar fracture.

The specimens were placed horizontally in the environmental chamber. The chamber has a 12-inch opening in the roof at 10 ½ feet from the chamber's entrance, built to accommodate an actuator extension for a previous large-scale column test conducted by Montejo et al., 2009a. The hole position was used as a reference to define the height of the column specimens for this study, which is 65 inches.

Figure 5-1 presents details of specimens 1 to 4. The column diameter is 18 inches with a 1-inch concrete cover, and the reinforcement consists of 10 #6 ASTM A706 grade 80 bars with #3 ASTM A706 grade 80 spirals at 2-inch spacing. An axial load ratio of 8% was selected. For columns 4 and 6, the transverse steel spacing was increased to 4 inches.

Table 5-2 Design Material Properties (Seismic Design Criteria Version 2.0, 2019)

Reinforcing steel material properties			
Property	Design Value		
Elastic Modulus	E= 29000 ksi		
Yield strength	Specified minimum,	$f_y = 80$ ksi	
	Expected,	$f_{ye} = 85$ ksi	
Tensile strength	Specified minimum,	$f_u = 98$ ksi	
	Expected,	$f_{ue} = 112$ ksi	
Yield strain	Nominal,	$\epsilon_y = 0.0028$	
	Expected,	$\epsilon_{ye} = 0.0033$	
Ultimate tensile strain	Nominal,	$\epsilon_u = 0.0950$	
	Expected,	$\epsilon_{ue} = 0.0600$	
Concrete material properties			
Property	Design Value	Equation	
Elastic Modulus	E= 4888 ksi	$E_c = 33w^{1.5} \sqrt{f'_{ce}}$, psi	
Compressive strength	Specified,	$f'_c = 5$ ksi	
	Expected,	$f'_{ce} = 6.5$ ksi	$f'_{ce} = 1.3 * f'_c$
Compressive strain	Unconfined strain at f'_c	$\epsilon_{co} = 0.002$	
	Ultimate unconfined strain (spalling)	$\epsilon_{sp} = 0.005$	

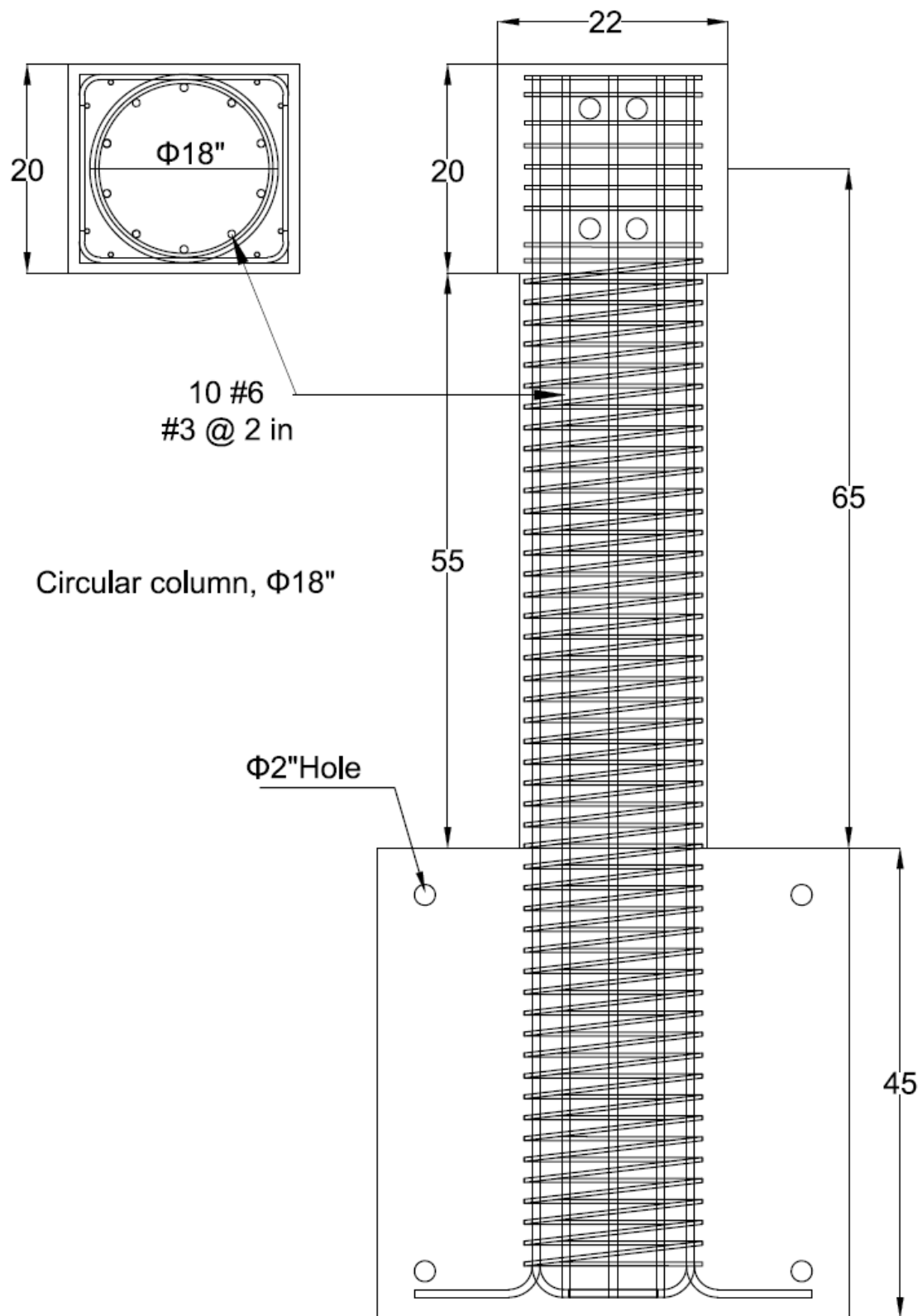


Figure 5-1 RC column specimen detailing (dimensions in inches)

5.2.2. Footing design

The footing was designed as a capacity-protected member using the column's overstrength moment input, ensuring that the footing yield moment was twice the column's overstrength moment. The environmental chamber has a pattern of eight pairs of holes, spaced every 3 feet, to allow access for bearing on the strong floor. The hole pattern defined the geometry of the footing, which was limited to 45 by 45 inches in plane, thus resulting in large shear demands and a heavily reinforced element. The design followed the beam-column joint design procedure presented in the book *"Seismic Design and Retrofit of Bridges"* by Priestley et al. (1996).

The footing's reinforcement consisted of 11 pairs of C-shaped #8 bars positioned in the parallel direction to the longitudinal steel of the column and 8 pairs of C-shaped #7 bars placed in the perpendicular direction. Sixteen 90-degree bars were positioned around the perimeter of the footing to prevent cracking around the PVC pipes. The 2-inch PVC pipes were placed to allow crossing bars for post-tensioning the footing to the strong floor, matching the hole pattern of the environmental chamber (3 ft by 3 ft). Additional shear reinforcement was provided with #3 J-hooks located every 6 inches in the perpendicular direction to the longitudinal steel of the column. The detailing of the footings is presented in Figure 5-2.

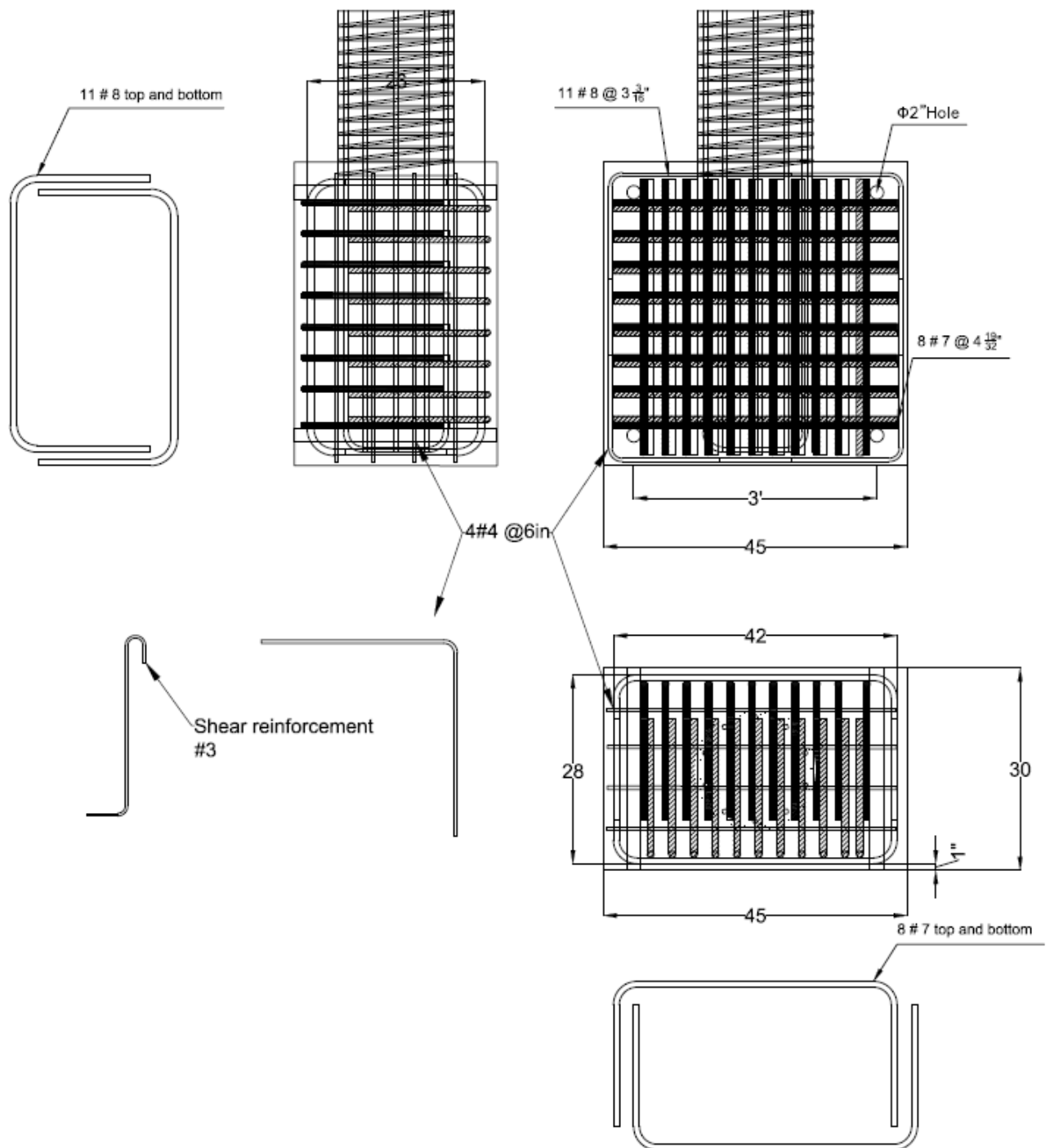


Figure 5-2 Footing detailing (dimensions in inches)

5.2.3. Construction process

The construction process for the large-scale specimens began with the construction of the column cages. The first step is to cut and bend the longitudinal reinforcing steel according to the detailing. For each column, ten straight #6 bars were cut to 192 inches and bent to an L shape with a 12-inch hook. Plywood templates were manufactured to accommodate the position of the reinforcing steel, providing external support for the bars at the ends and internal support along the span. The templates at each end were secured to two concrete blocks using ratchet straps to keep them in place while providing a comfortable height for the construction. The spirals were supported at the column level with an overhead crane while the longitudinal bars were positioned. Wooden spacers were used to ensure the spacing of the spirals (center-to-center), while the longitudinal bars were tied to the spirals with wire at each intersection. Due to the limitation of space in the footing, three pairs of hooks on the longitudinal bars were connected inwards to the column diameter, and the other two pairs were placed outwards. Figure 5-3 illustrates column cage construction.



Figure 5-3 Column cage construction

The next step in the construction process involved assembling the footing cage and casting the footing concrete. The footing formwork was constructed using a steel-ply forming system comprising three 30-inch by 4-foot panels, four 24-inch by 4-foot panels, six 4-foot steel angles, and two 3-foot steel angles per footing, all connected with wedge bolts. A 3-inch-thick foam block was installed in the formwork to ensure the footing's correct geometry. Four-panel form ties were added to each footing formwork to prevent the formwork from opening under the pressure of the concrete. In addition, four 2-inch PVC pipes were drilled into the formwork to create openings for positioning the footing relative to the footing support and strong floor. The column cage was placed in the formwork, and the footing cage was built around it, as shown in Figure 5-4.

Once the footing cage was ready, the next step was to close the formwork and prepare for concrete pouring. This process included applying a form release agent to the formwork and ensuring the column was plumbed vertically. Figure 5-5 shows the footings ready for casting, with the columns fixed in position and the footings being cast. Concrete cylinders were cast to monitor the strength development of the concrete used for the footings.



Figure 5-4 Footing cage construction and formwork



Figure 5-5 Footing's concrete casting

After the footings were built, the columns and cap beams were constructed. Scaffolding was erected around the column cages to support the cap beam casting and provide access to the top of the specimen, facilitating concrete pouring and vibration. Two specimens were positioned within each scaffolding setup. Figure 5-6 shows the specimen arranged within the scaffolding in preparation for the column and cap beam construction.



Figure 5-6 Placement of the specimens and scaffolding for column and cap beam construction

Two type T thermocouples were installed in each specimen: one to measure the temperature at the core of the column and the other at a longitudinal bar. Additionally, 1-inch-thick foam blocks were placed around the perimeter of the reinforcing steel along the expected plastic hinge length to expose the steel in that area (without cover) after casting. Exposing the steel in the plastic hinge region facilitates the installation of the column's instrumentation and enables tracking of the damage progression on the reinforcing steel during the test.



Figure 5-7 Thermocouples installation



Figure 5-8 Foam blocks used to expose reinforcing steel along the plastic hinge.

The column formwork consisted of an 18-inch (internal diameter) cardboard concrete-forming tube cut to the column's clear length (55 inches). A hole was cut in the forming tube to allow the thermocouple cables to pass through. The cap beam's formwork was built using $\frac{3}{4}$ -inches thick plywood and pressure-treated lumber. Two-inch holes were drilled in the cap beam formwork using a hole saw to cross PVC pipes for the cap beam-actuator connection. Additionally, a wooden frame was built from 4x4-inch pressure-treated lumber to provide vertical support for the scaffolding, accounting for the weight of the formwork and concrete, as well as the loads imposed during casting. The concrete forming tube was plumbed and fixed to the wooden frame using 2x4-inch pressure-treated lumber, drilled into the frame, and secured with ratchet straps. The column and cap beam formwork are shown in Figure 5-9.

After placing the wooden frame and formwork, the reinforcing steel for the cap beam and the PVC pipes were installed, and the connection between the column form and the cap beam was sealed with waterproof silicone sealant (Figure 5-10). Once the formworks were sealed, the columns and cap beam were ready to receive concrete. Figure 5-11 shows the column and cap beam casting.



Figure 5-9 Formwork and frame support placement



Figure 5-10 Placement and sealing of cap beam steel and PVC pipes, and placement of formwork



Figure 5-11 Column and cap beam concrete casting

Once the concrete had set, the formwork, PVC pipes, and foam blocks were removed, as seen in Figure 5-12 and Figure 5-13. The specimen was then prepared for instrumentation and testing. This process involved cleaning the exposed reinforcing steel along the plastic hinge using wire brushing and removing excess concrete with a chisel and an air hammer. A strip disc was used to smooth the surface of the longitudinal bars where strain gages would be placed. A drill hammer was used to drill 3/8-inch holes for threaded rods to support the linear potentiometers (further details on this system are provided in the instrumentation section), and the column was painted with an eggshell white, water-based paint to facilitate crack marking along the test. Figure 5-14 shows the exposed bars cleaned of concrete excess, the smooth surfaces on the longitudinal steel for the strain gages, and the 3/8-inch drilled holes for the threaded rods used to support the linear potentiometers.



Figure 5-12 RC column specimens and removal of the formwork



Figure 5-13 Removal of the formwork



Figure 5-14 Cleaning of exposed reinforcing steel and column preparation for instrumentation

5.3. Large-scale testing methods

5.3.1. Experimental setup

The experiments were conducted in the Constructed Facilities Laboratory (CFL) at NC State University. The tests were conducted inside a walk-in Thermotron environmental chamber, model WP-1512-CHM-25-25, measuring 18 ft x 8 ft x 8 ft, with a temperature range from -90 °F to 185 °F (-68 °C to 85 °C). Due to the geometric constraints of the environmental chamber, the specimens were designed to be positioned horizontally. Therefore, the cyclic load was applied vertically to simulate lateral forces on the column. The columns were fixed in a cantilever configuration to allow top displacement during pushing cycles. A metallic base elevated the footing 18 inches above the chamber floor, while four post-tensioned bars secured the footing and base to the strong floor, ensuring a fixed connection.

The test setup consisted of a structural steel test frame that supported a 110-kip MTS actuator with a 20-inch stroke, which applied cyclic loading to the column. The actuator was connected to an actuator extension that crossed the environmental chamber roof through a 12-inch hole to transmit the load to the column. The actuator extension was connected to a steel shoe attached to the column cap beam with a 2-inch pin, allowing rotation in the column's parallel direction. The steel shoe was attached to the cap beam using threaded rods, as shown in Figure 5-15. Montejo et al. (2009a) fabricated the actuator extension to conduct prior research on the seismic performance of RC columns at low temperatures using ASTM A706 grade 60 steel.

To maintain consistent testing conditions, the same setup was used for the room and low-temperature specimens. The only difference is that low-temperature tests were conditioned and tested inside the environmental chamber to maintain the target temperature, while room-temperature tests were not.

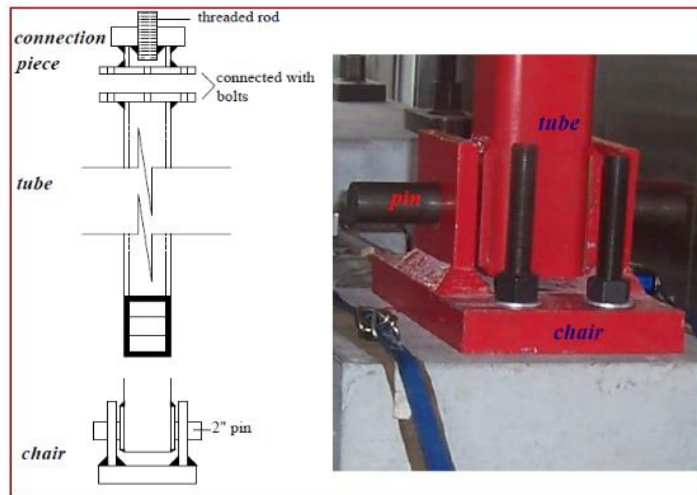


Figure 5-15 Actuator extension by Montejo et al., 2009a

During all tests, a constant axial load of 123-kip was applied using two double-acting hollow-plunger hydraulic cylinders (RRH10010), two spreader beams, and two 1 3/8-inch threaded bars. The hydraulic cylinders were located at the top of the columns and supported

over a spreader beam. The jacks were powered directly by the 3000-psi hydraulic system at the CFL, continuously throughout the test. Two 1 3/8-inch threaded bars transmitted the axial load to the column connecting the top and bottom spreader beams. The general layout of the experimental setup is shown in Figure 5-16. A closer look at the setup inside the chamber is shown in Figure 5-17.

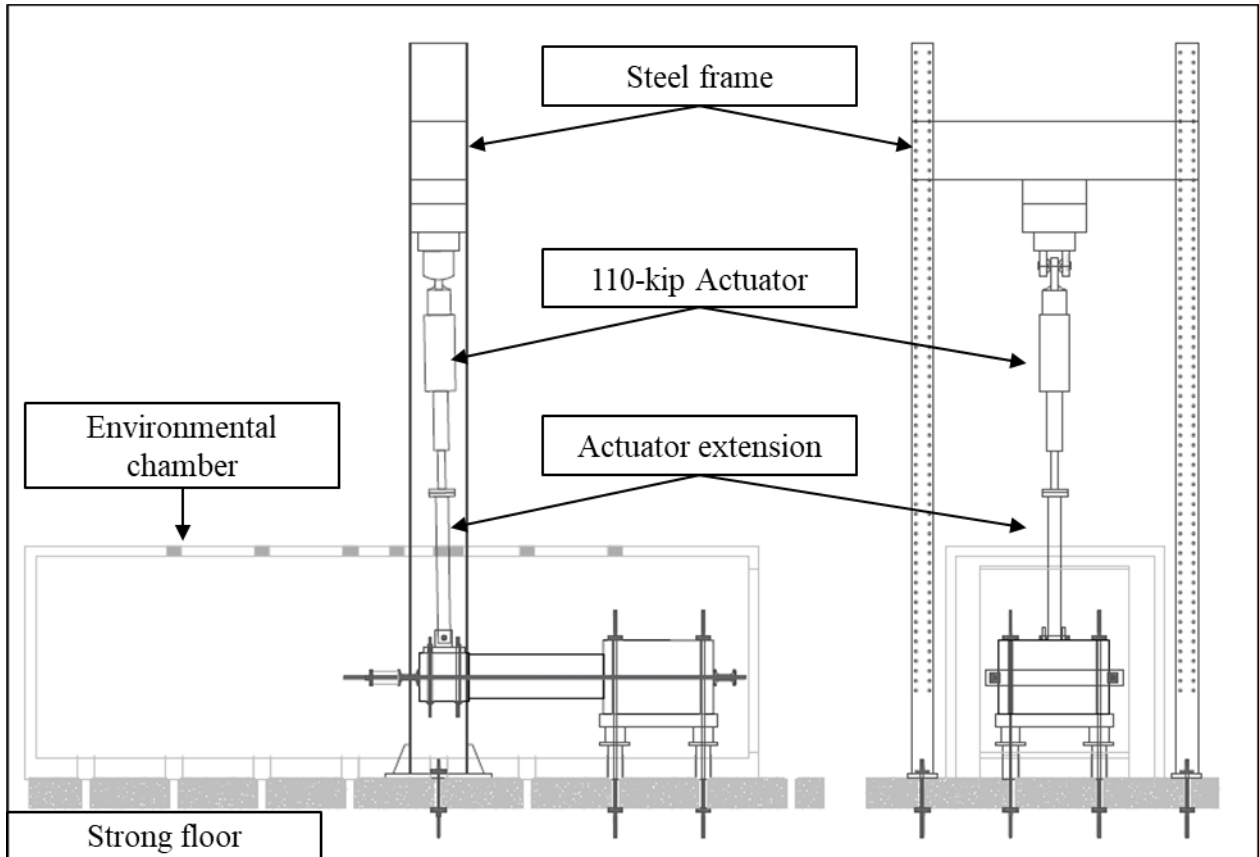


Figure 5-16 Experimental setup (general layout).

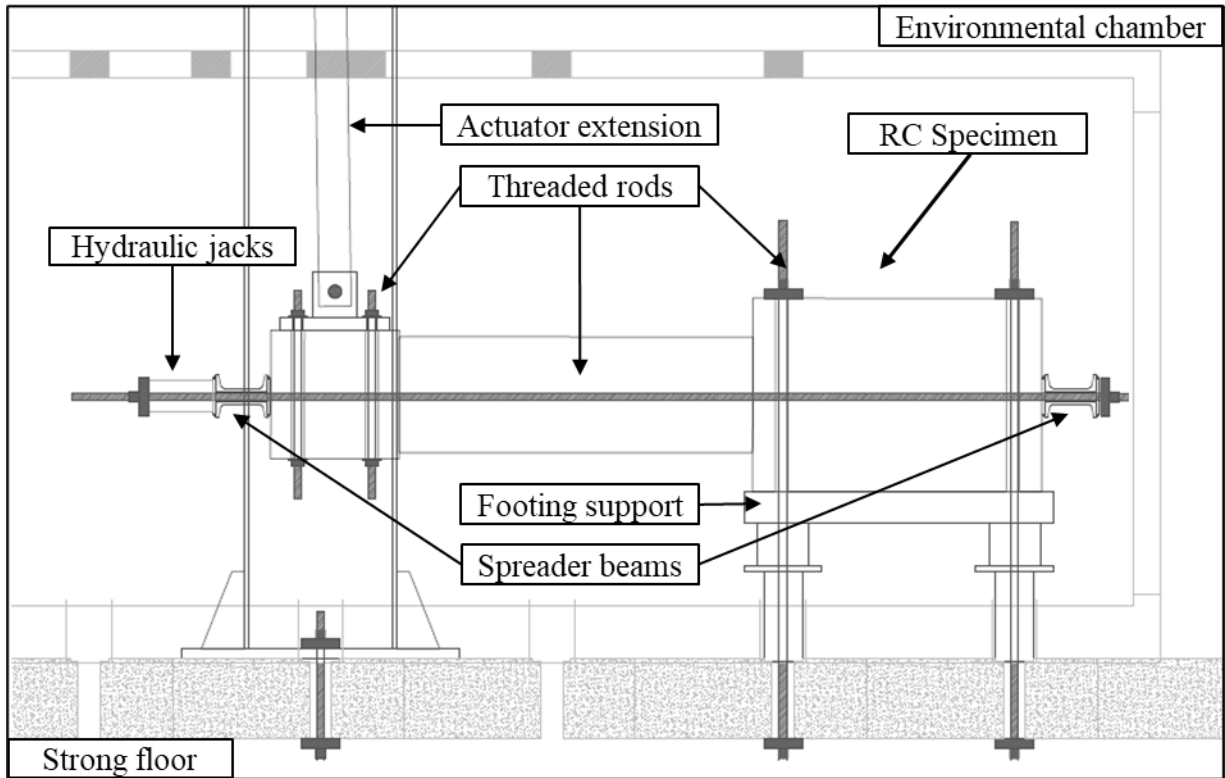


Figure 5-17 Experimental setups (view inside the environmental chamber)

5.3.2. Loading protocol

The loading protocol was defined based on the first test, which involved an RC column reinforced with MA steel tested at room temperature. The cycles of lateral loading were determined in two parts: the first portion of the loading protocol was a "force control" range, in which displacements were applied to achieve a specific load. This range corresponds to the first 4 cycles, where each cycle was defined to achieve proportional values to the predicted first yielding force as $1/4 F_y$, $1/2 F_y$, $3/4 F_y$, and F_y (Table 5-3). The second range was defined as a "displacement control" range (Table 5-5). In the displacement control range, a set of displacements was determined to represent different ductility levels: 1, 1.5, 2, 3, 4, 5, and 6, each proportional to the equivalent yield displacement. The equivalent yield displacement of the column, Δ_y (Table 5-4), was obtained using Eq. (6), where M_n is the predicted nominal moment, M_y is the predicted first yielding moment, and Δ_y is the average

of the displacements obtained experimentally from the pulling and pushing cycle at F'_y . The predictions were obtained through a sectional analysis. The analysis used the MATLAB CUMBIA code set (Montejo et al. 2007) with expected material properties. Three complete loading cycles were applied for each ductility level. Figure 5-18 presents the loading protocol applied to the column. In addition, a constant axial load of 123 kips was applied to the columns to simulate the vertical loads of a bridge column for a designed ALR of 8%.

$$\Delta_y = \Delta'_y \frac{M_n}{M'_y} \quad \text{Equation (6)}$$

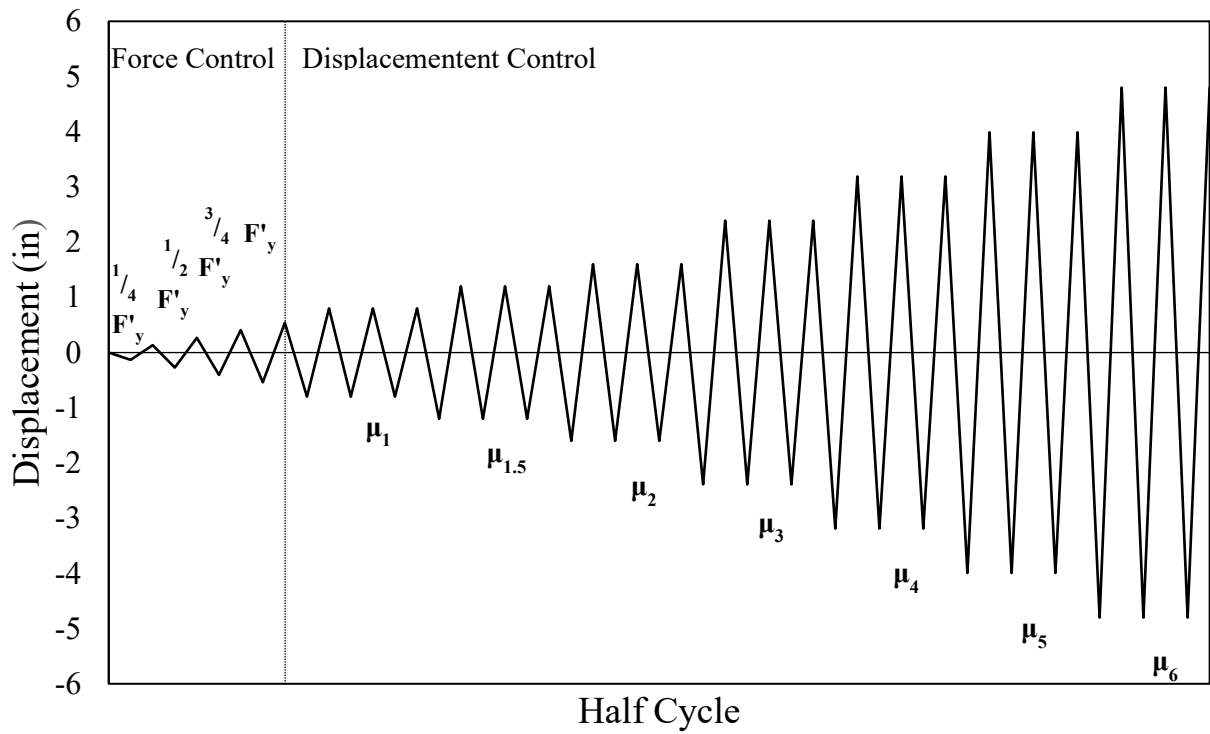


Figure 5-18 Cyclic loading protocol definition

Table 5-3 Load protocol definition - Force control range

Force control range			
State	Force		Cycles
1/4 F'_y	8.25	kip	1
1/2 F'_y	16.5	kip	1
3/4 F'_y	24.8	kip	1
F'_y	33	kip	1

Table 5-4 Calculation of equivalent yield displacement.

F'y (kip)	- Δ'y (in)	+ Δ'y (in)	Average Δ'y (in)	Δy (in)
33	0.57	0.58	0.58	0.8

Table 5-5 Load protocol definition - Displacement control range.

Displacement control range			
Ductility	Displacement	Cycles	
μ1	0.8 in	3	
μ1.5	1.2 in	3	
μ2	1.6 in	3	
μ3	2.4 in	3	
μ4	3.2 in	3	
μ5	4.0 in	3	
μ6	4.8 in	3	

5.3.3. Instrumentation

Each column was instrumented using two thermocouples, thirty strain gages, twelve linear potentiometers, six-string potentiometers, and two inclinometers. The columns tested at room temperature also used optical instrumentation systems to compare and validate the deformations obtained from the other instruments.

The embedded thermocouples were used to monitor temperatures in the column's core and the longitudinal steel during testing. Strain gauges were used to measure strains in the steel along the length of the plastic hinge. Linear potentiometers were employed to track relative axial elongation within a defined 6-inch gauge length, helping to assess curvature along the plastic hinge and strain penetration in the column. String potentiometers measured the top displacement and the deformed shape along the plastic hinge. Lastly, the inclinometers were used to determine any rotation of the footing and cap beam around the direction perpendicular to the column.

All instruments were connected to a StrainSmart Data Acquisition System 7000 for data collection during testing. Data recording commenced once the specimen reached a stable set temperature to eliminate errors caused by the instrument's thermal contraction, and after the column was lifted to a zero position, to counteract its self-weight and the weight of the beam and jacks installed at the top of the column for the axial load system. Figure 5-19 illustrates the instrumentation setup on an RC column specimen.

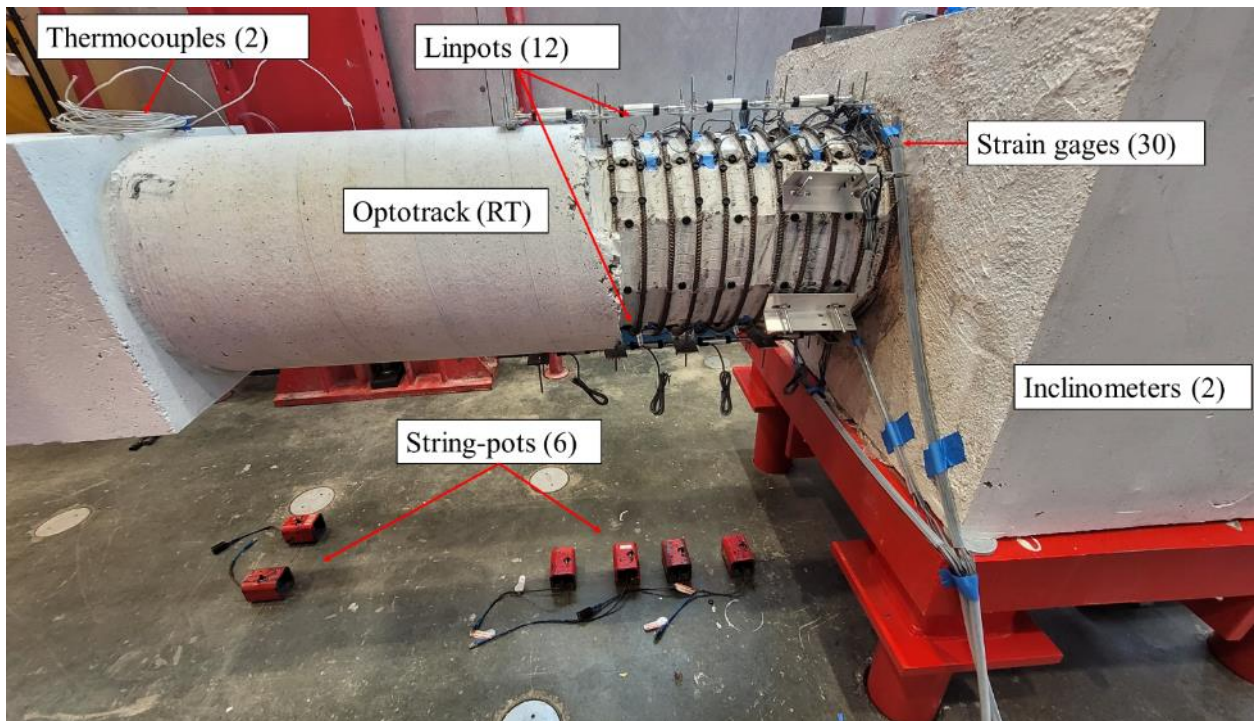


Figure 5-19 Instrumentation of the RC column specimen

5.3.4. Material testing

Reinforcing steel: Two manufacturing processes, MA and QST, were evaluated using large-scale testing. The straight QST bars were donated by Commercial Metals Company (CMC), and Nucor Corporation donated the straight MA bars and the #3 coil for the spirals. The spirals were fabricated by Steel Specialties of Mississippi and shipped to the CFL at NC State University.

Tension tests at room temperature were performed on the longitudinal steel used to construct the column specimens. Three #6 bars were tested for each manufacturing process to determine their corresponding tensile properties. Elongation was recorded using a 2-inch gage Epsilon. The extensometer was removed at each test when the bar reached maximum stress to prevent damage to the equipment caused by bar rupture. Therefore, the stress-strain curves are presented until the peak stress.

The mechanical properties were obtained in accordance with the definitions presented in ASTM A370-20. The yield strength was determined using the 2% offset method specified in ASTM A370-20. The modulus of elasticity was obtained from the slope of the linear portion of the stress-strain curve, and the yield strain was obtained by dividing the yield stress by the elastic modulus. The peak stress corresponded to the maximum applied force on the bar, and the uniform elongation corresponded to the elongation measured at the peak stress. A summary of the tensile properties obtained from the tension test and a comparison with the ASTM A706 provisions are presented in Table 5-6. The tensile properties of the spirals are shown in Table 5-7.

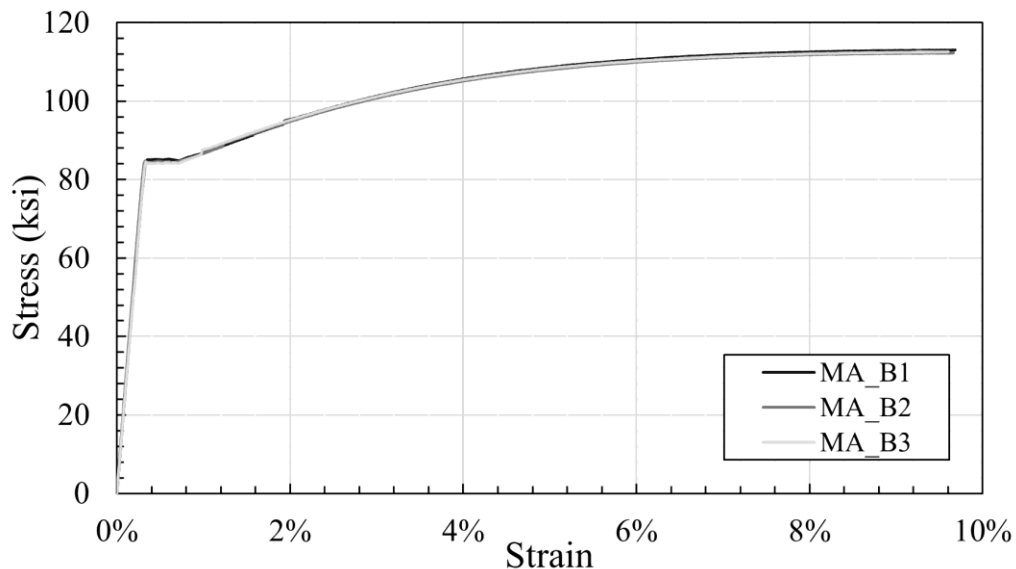


Figure 5-5-20 Stress-strain curve of MA bars used as longitudinal steel in the RC specimens

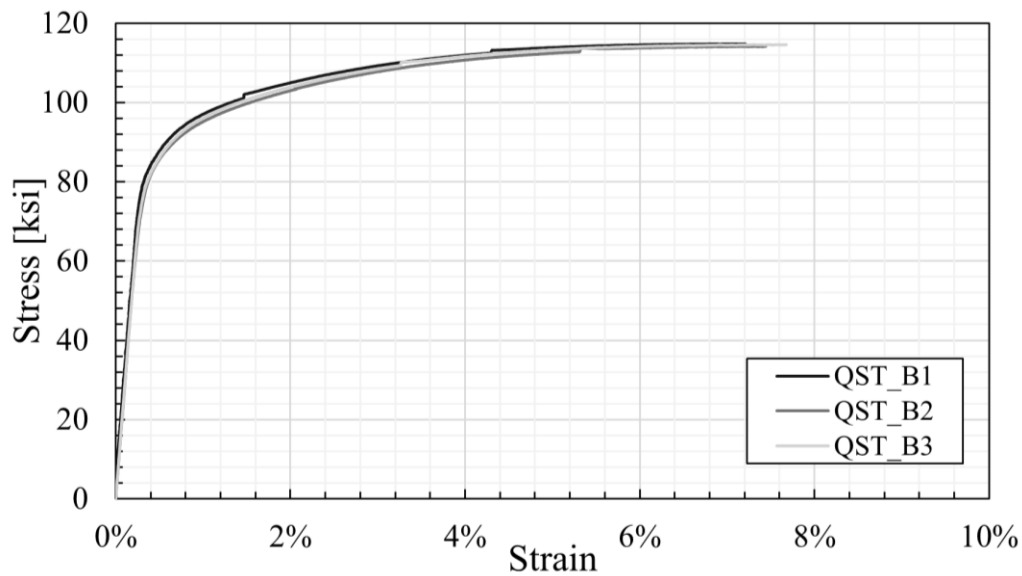


Figure 5-21 Stress-strain curve of QST bars used as longitudinal steel in the RC specimens

Table 5-6 Tensile properties of the longitudinal steel

Bar specifications			Fy (ksi)	E (ksi)	ϵ_y	Fu (ksi)	ϵ_u	T/Y
ASTM A706-22 #6 bar	min		80			100	7.0%	1.25
	max		98					
MA (Average)			85	26259	0.32%	113	9.7%	1.33
QST (Average)			86	27235	0.32%	115	7.4%	1.33

Table 5-7 Tensile properties of the transverse steel

Bar specifications			Fy (ksi)	E (ksi)	ϵ_y	Fu (ksi)	Elongation in 8-inch	T/Y
ASTM A706-22 #3 bar	min		80			100	12%	1.25
	max		98					
Spirals #3 (MA)*			91	27000	0.34%	123	12%	1.35

*This data was provided by the coil manufacturer (NUCOR)

Concrete: Concrete strength was specified at 5 ksi for all castings. Cylinders with a diameter of 4 in and a height of 8 in from column and footing castings were used to determine the maximum compressive strength on the test day of each specimen. For the columns tested at low temperatures (Cols 3-6), concrete cylinders were conditioned at -40°F (-40°C) and tested outside the environmental chamber. Table 5-8 presents the concrete strength obtained

from the cylinders corresponding to all the column castings. Compression tests of the concretes were performed within a week of each column test day.

Table 5-8 Concrete compressive strength of the RC column specimens

Col #	Column Specification	Temperature	Concrete age (test day)	F' _c (ksi)	Average (ksi)
Col 1	MA-RT-2in	RT	193	5.97	6.24
				6.69	
				6.06	
Col 2	QST-RT-2in	RT	231	6.95	6.98
				7.22	
				6.77	
Col 3	MA-LT-2in	RT	329	6.68	6.68
Col 3	MA-LT-2in	LT	329	6.68	9.50
				9.45	
				9.56	
Col 4	QST-LT-2in	RT	425	7.15	7.14
Col 4	QST-LT-2in	LT	425	7.13	9.34
				8.95	
				9.72	
Col 5	MA-LT-4in	RT	57	4.42	4.45
				4.53	
				4.40	
Col 5	MA-LT-4in	LT	57	7.56	7.82
				7.95	
				7.94	
Col 6	QST-LT-4in	RT	110	4.84	4.91
				4.95	
				4.95	
Col 6	QST-LT-4in	LT	110	7.86	7.37
				6.58	
				7.66	

RT: Concrete cylinders tested at room temperature

LT: Concrete cylinders tested after low temperature conditioning at -40°F for more than 20 hours.

5.3.5. Column predictions

As mentioned earlier, the test predictions were obtained through a sectional analysis using the MATLAB CUMBIA code set (L. A. Montejo, 2007). The predictions were conducted using the material properties presented from Table 5-6 to Table 5-8. For the low-temperature predictions, the yield and ultimate strength of the transverse and longitudinal steel were multiplied by 1.07 to account for the 7% strength increase at -40°F (-40°C) reported in the material-level evaluation for grade 80 steel. The compressive strength of the cylinders tested after low-temperature conditioning was also used.

The sectional analysis assumes the axial load acts at the center of the section, ignoring P- Δ effects. Therefore, a correction was applied to the force response to reduce the capacity due to second-order effects. The P- Δ correction accounted for the deformed geometry representation of the RC column specimen shown in Figure 5-22. Where Δ corresponds to the relative displacement between the axial load bar and the center of the bottom section of the column, and P corresponds to the horizontal component, P_{axial}, of the applied compressive force. Δ and P_{axial} were calculated at each top displacement step within the analysis. Eq. (7) was used to find Δ .

The total moment obtained from the moment-curvature analysis has two main components (Eq. (8)). The first corresponds to the moment due to the lateral force. And the second, to the moment, due to the P- Δ effect. The corrected force is then obtained using Eq. (9). The corrected force-displacement response is compared later in this report with the experimental force-displacement response for each test. Figure 5-23 compares the force-displacement obtained directly from the moment-curvature analysis with the one obtained after correcting the force for P- Δ effects of Column 1 (MA_RT_2in). Higher capacity is obtained when P- Δ effects are ignored.

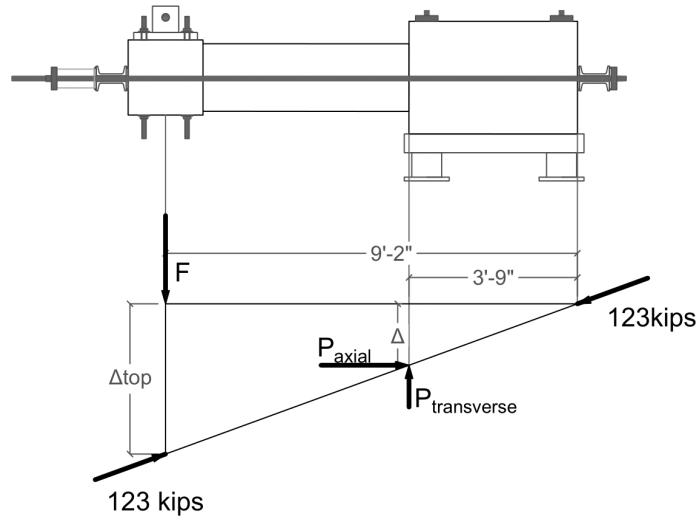


Figure 5-22 Geometric representation of the deformed RC specimen to account for P-Δ effects

$$\Delta = \frac{\Delta_{top} * 45}{110} [in] \quad \text{Equation (7)}$$

$$M = FL_{col} + P_{axial}\Delta \quad \text{Equation (8)}$$

$$F = \frac{M - P_{axial}\Delta}{L_{col}} \quad \text{Equation (9)}$$

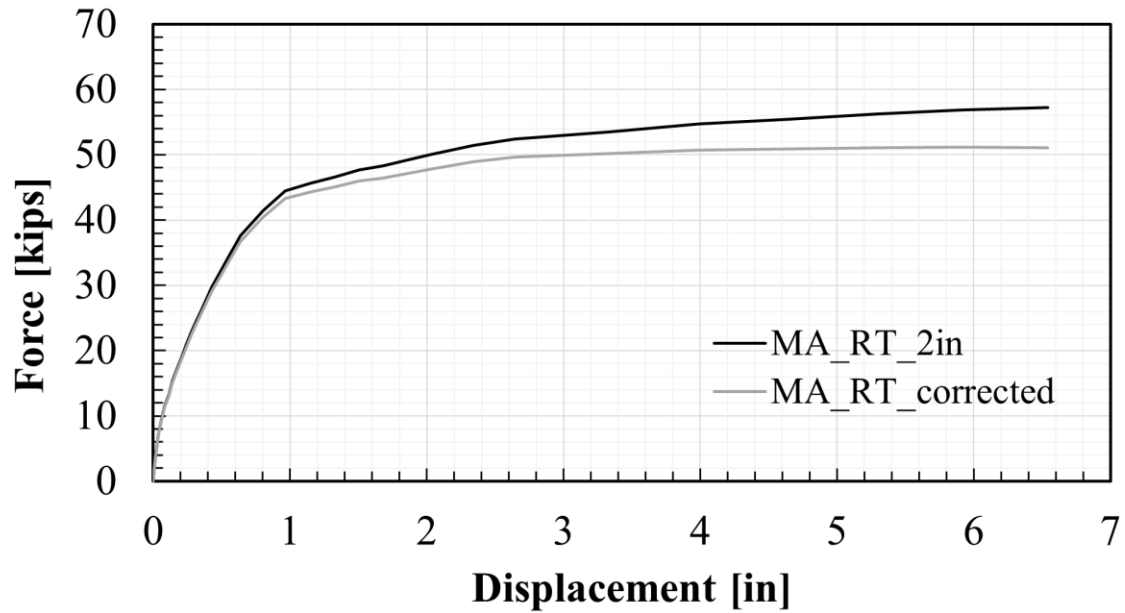


Figure 5-23 P-Δ effects on the Force-Displacement predictions

5.4. Experimental response COL1: MA RT 2IN

5.4.1. Summary

This section summarizes the experimental response from the first column test. The test was performed at room temperature on an 18-inch-diameter column reinforced with 10 #6 Micro-alloyed ASTM A706 grade 80 bars. The specimen reached a maximum force of 50 kips and exhibited a 20% strength loss during the third pushing cycle of $\mu 5$ and the first pulling cycle of $\mu 6$, due to the fracture of the extreme bars. The column was loaded until the second cycle of $\mu 6$, at which point it lost approximately 70% of its strength. A summary of the column parameters is presented in Table 5-9.

This test served as a baseline to define the loading protocol presented earlier, which was used in all column tests to maintain a constant displacement demand input along the tests.

Table 5-9 Summary of column parameters

Column parameters	
Longitudinal Steel:	10 #6 ASTM A706 Gr.80 ($\rho_l=1.74\%$) MA
Transverse Steel:	#3 ASTM A706 Gr.80 @ 2" ($\rho_t=1.23\%$)
Longitudinal Steel Nominal Yield Strength:	84.5 ksi
Transverse Steel Nominal Yield Strength:	91.2 ksi
Concrete Strength:	6.3 ksi
Axial Load:	123 kips
Axial Load Ratio:	7.7%
First Yield Displacement:	0.57 in
F _y	36 kip
Nominal Moment Capacity:	250 kip-ft
Equivalent Yield Displacement:	0.8 in
Maximum Lateral Force:	50.4 kips
Displacement at 20% Strength loss:	4 in
Failure Mode:	Fracture of buckled bar

5.4.2. Test observations

Table 5-10 presents the summary of damage observations from the MA_RT_2 in the column test. Starting from the cycle at $\frac{1}{2} F_y$, hairline cracks formed along the plastic hinge and continued expanding throughout the test. In the last cycle of μ_3 , the concrete began to flake. The extreme top bar (2T) buckled first during the third pulling cycle of μ_4 and fractured during the third pushing cycle of μ_5 . The extreme bottom bar (2B) buckled at approximately the third pushing cycle of μ_5 and fractured in the first pulling cycle of μ_6 . In the second pushing cycle of μ_6 , the bars 1T and 3T fractured. At this point, the test was finalized. Figure 5-24 to Figure 5-35 illustrate the progression of damage to Column 1 throughout the test.

Table 5-10 Column 1 damage progression summary.

Damage observations	
Initiation of cracking:	$\frac{1}{2} F_y$
Core Concrete Crushing:	μ_3^{+3}
Longitudinal Bar Buckling Top:	μ_4^{+3}
Longitudinal Bar Buckling Bottom:	μ_5^{-3}
Longitudinal Bar Fracture Top:	μ_5^{-3}
Longitudinal Bar Fracture Bottom:	μ_6^{+1}
20% Strength loss (-):	μ_5^{-3}
20% Strength loss (+):	μ_6^{+1}
(+) Pulling, (-) Pushing	



Figure 5-24 The first cycle of first yielding (pulling).

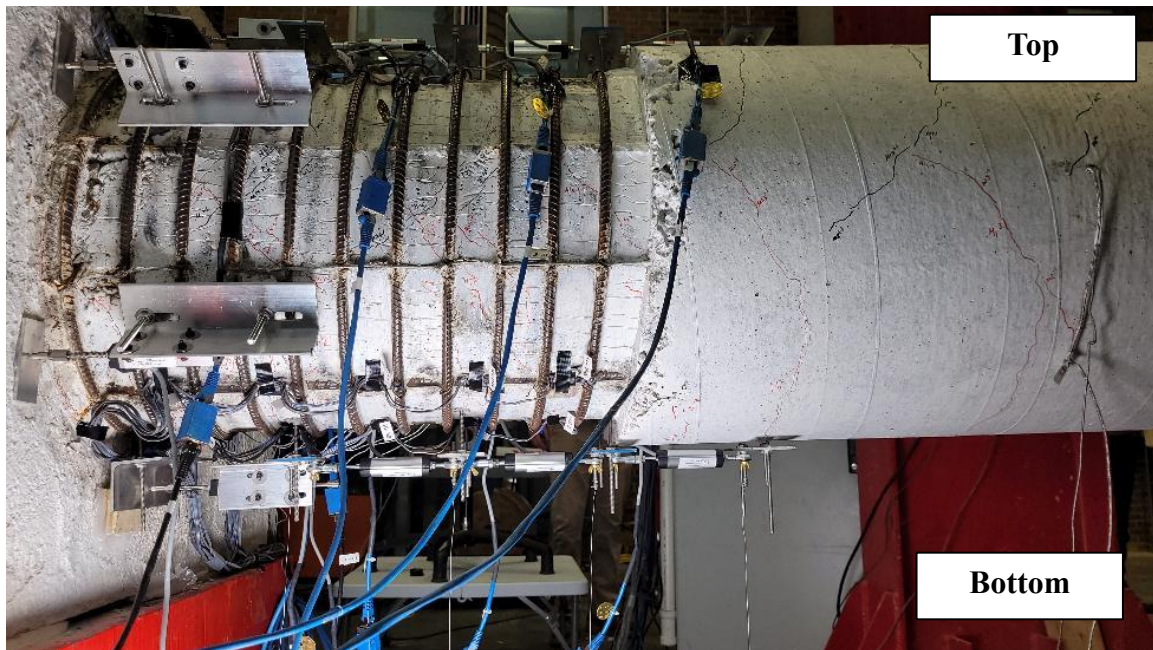


Figure 5-25 First cycle μ_2 (pulling). Cracks form and expand along the plastic hinge. Some hairline shear cracks were observed along the column.

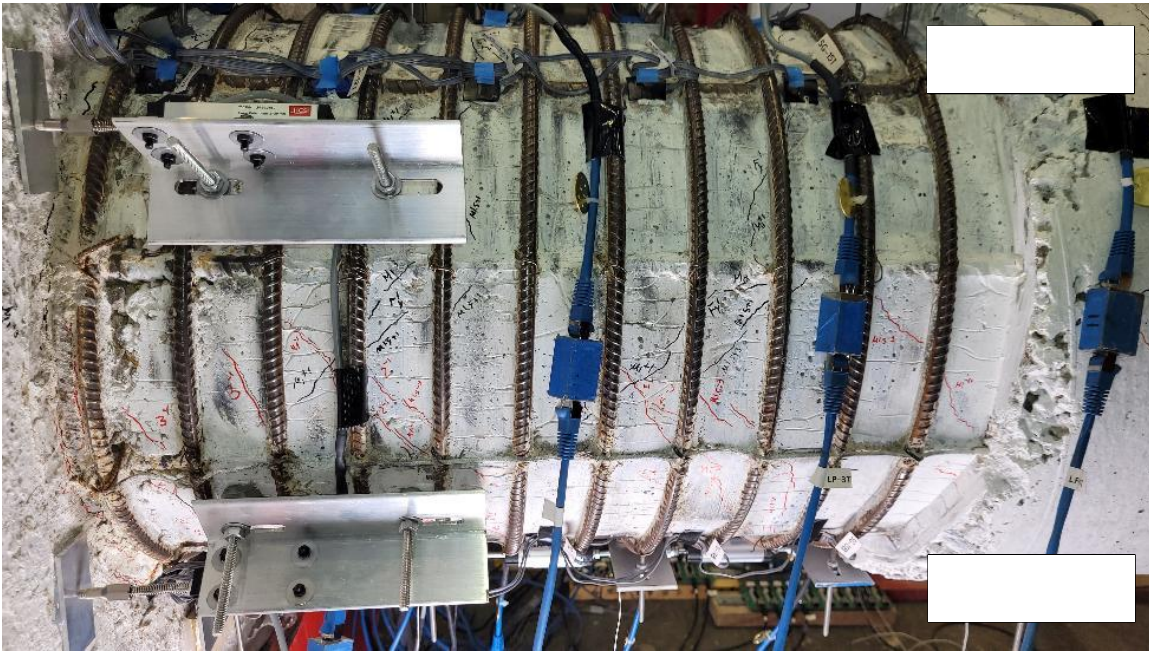


Figure 5-26 First cycle μ_3 (pushing): Cracks at the base of the column and along the plastic hinge expand.



Figure 5-27 First cycle μ_3 (pushing): Extension of cracks at the base of the column and in the footing.

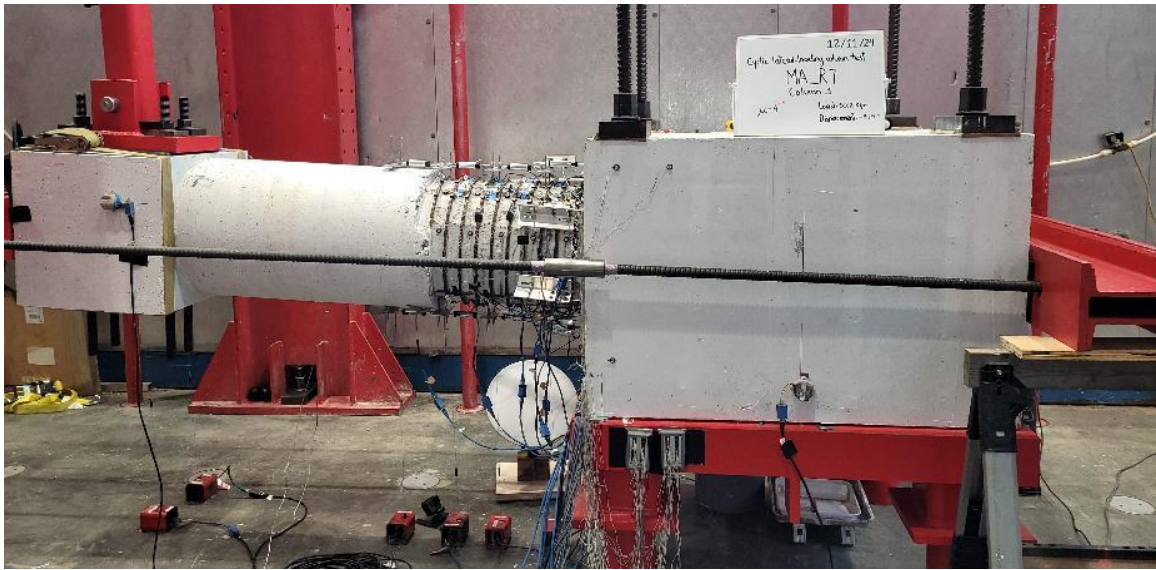


Figure 5-28 First cycle of μ_4 (pulling).



Figure 5-29 First cycle of μ_4 (pulling): Concrete crushing around bar T2, and cracking and uplifting of concrete at the footing.



Figure 5-30 Third cycle of μ_4 (pulling): Buckling of the top extreme bar 2T and concrete crushing



Figure 5-31 Third cycle μ_5 : Visible bar buckling in pulling cycle and fractured in pushing cycle of the top extreme bar 2T



Figure 5-32 First cycle μ_6 (pulling)

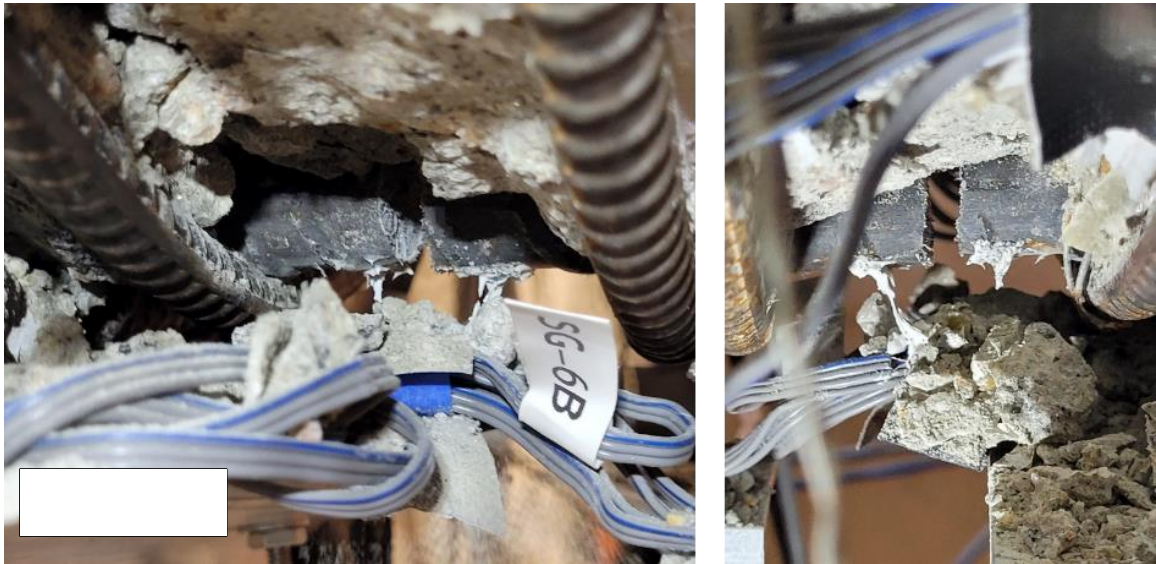


Figure 5-33 First cycle μ_6 (pulling): Fracture of bottom extreme bar, 2B.



Figure 5-34 Final state of the specimen: Fracture of bars 1T, 2T, and 3T, and concrete core crushing.



Figure 5-35 Damage at the bottom of the column: Fracture of bar 2B, concrete core crushing, and footing concrete breakout due to stress concentrations on footing support.

5.4.3. Force-displacement response

The hysteretic response of the column is shown in Figure 5-36. The specimen exhibited fractures in four bars, as indicated by an X on the hysteretic response. The first bar fractured during the last pushing cycle of μ_5 , which corresponded to the extreme top bar (2T). At this point, a 20% drop in strength was observed in the pushing direction (-) due to the bar fracture. Similarly, a 20% drop in strength was observed in the pulling direction (+) after the bar fracture of the extreme bottom bar (2B), which occurred at the first pulling cycle of μ_6 . The two bars adjacent to the extreme top bar (1T and 3T) fractured at the second pushing cycle of μ_6 , at which point a drop in strength of about 70% of the maximum force was observed. This damage progression is also evident in Figure 5-37, which shows the force-displacement envelope response of the column. These envelopes of the experimental results are compared with the predictions from the sectional analysis, accounting for P- Δ effects, and it is shown that the experimental results align well with the predictions.

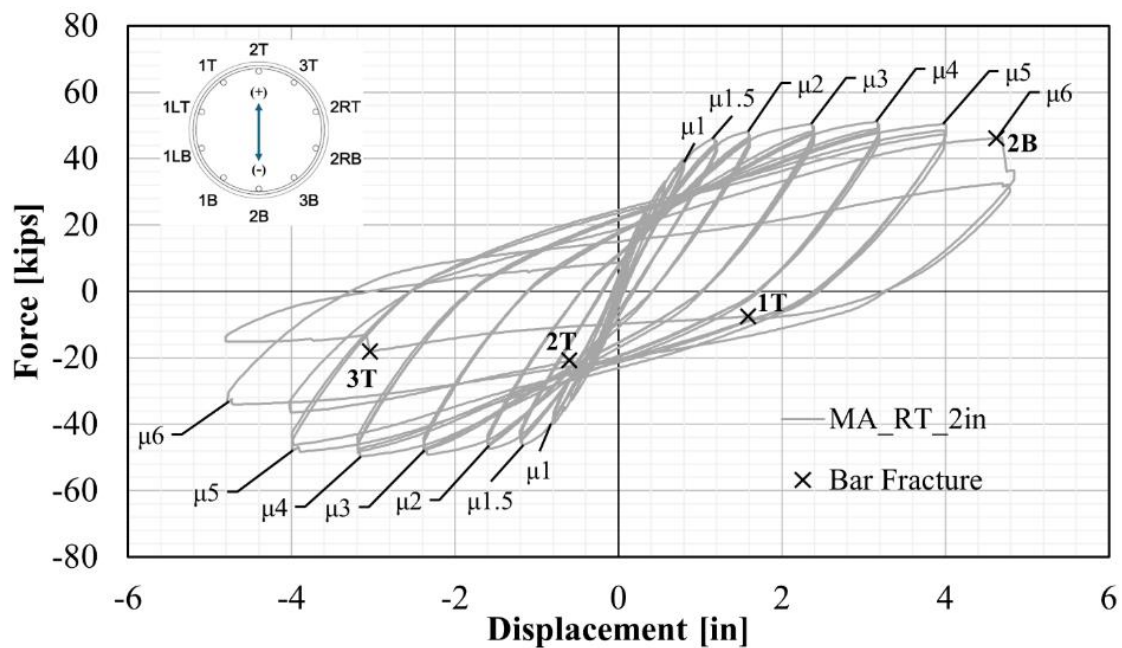


Figure 5-36 Hysteretic response of Column 1: MA_RT_2in

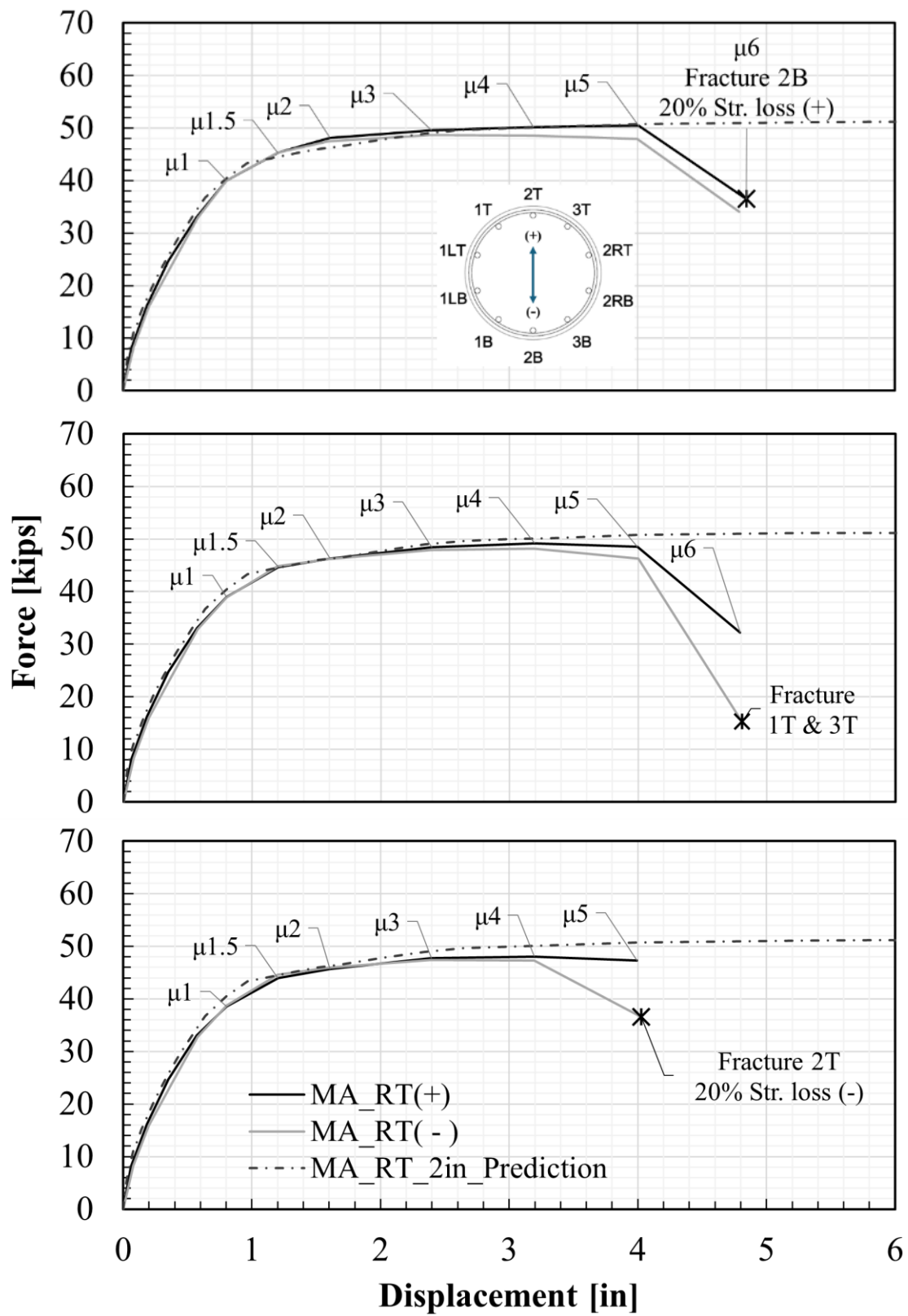


Figure 5-37 Envelope of the Force-displacement response of Column 1: MA_RT_2in.

5.4.4. Curvature Profile

The curvature profile was calculated using the rotations obtained from the relative axial displacements measured by the linear potentiometers mounted on the sides of the columns (See Figure 5-19). Four linear potentiometers were aligned with the extreme bars on the column cross-section. The linear potentiometers measured relative axial displacement over a gage length of approximately 7 inches, yielding a coarse curvature profile, with each marker representing the average curvature over the gage length containing the potentiometer.

Figure 5-38 and Figure 5-39 show the low-level (up to first yield of the column section) and high-level (beyond first yield of the column section) curvature profile of Column 1: MA_RT_2in, respectively. The predicted first yield curvature is also presented, assuming a linear distribution from the base of the column to the top of the column. In Figure 5-38 a symmetric distribution of curvature is noted for each direction of loading. A larger concentration of curvature is evident at the bottom of the column.

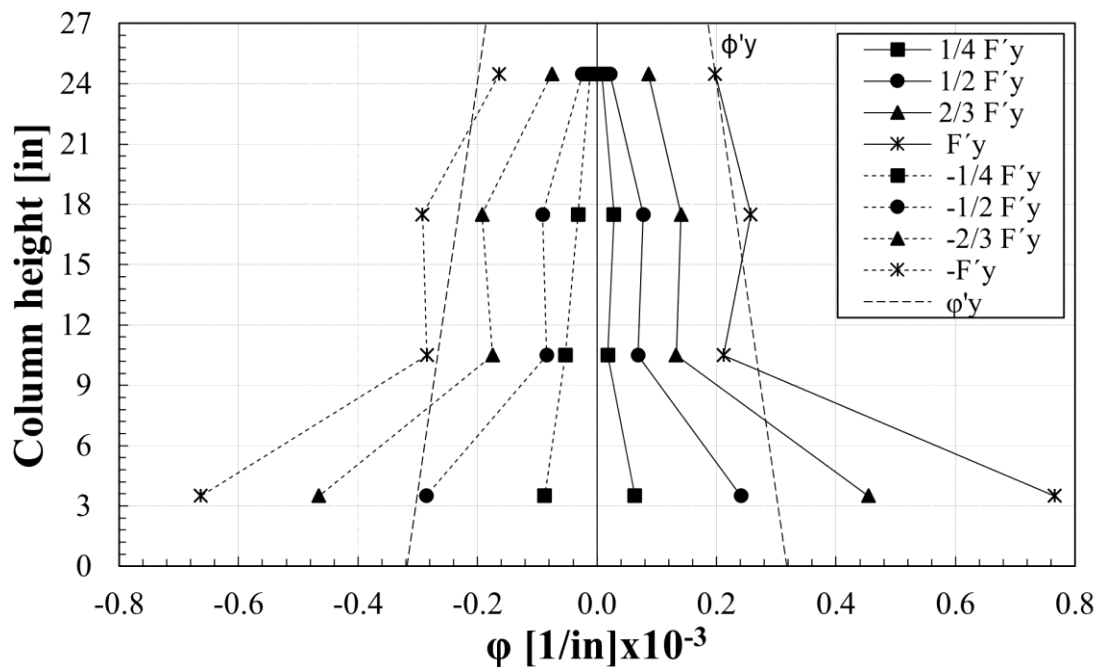


Figure 5-38 Low-level curvature profile of Column 1: MA_RT_2in

Similarly, Figure 5-39 shows a symmetric distribution for the high-level curvatures, with slightly higher values in the pulling direction. The curvature data for μ_6 in the pushing direction (dashed lines) were not presented due to data uncertainty, as the instrument support at the base of the column was compromised due to column damage. Overall, the larger curvatures were concentrated near the base of the column, and this observation was consistent across different ductility levels.

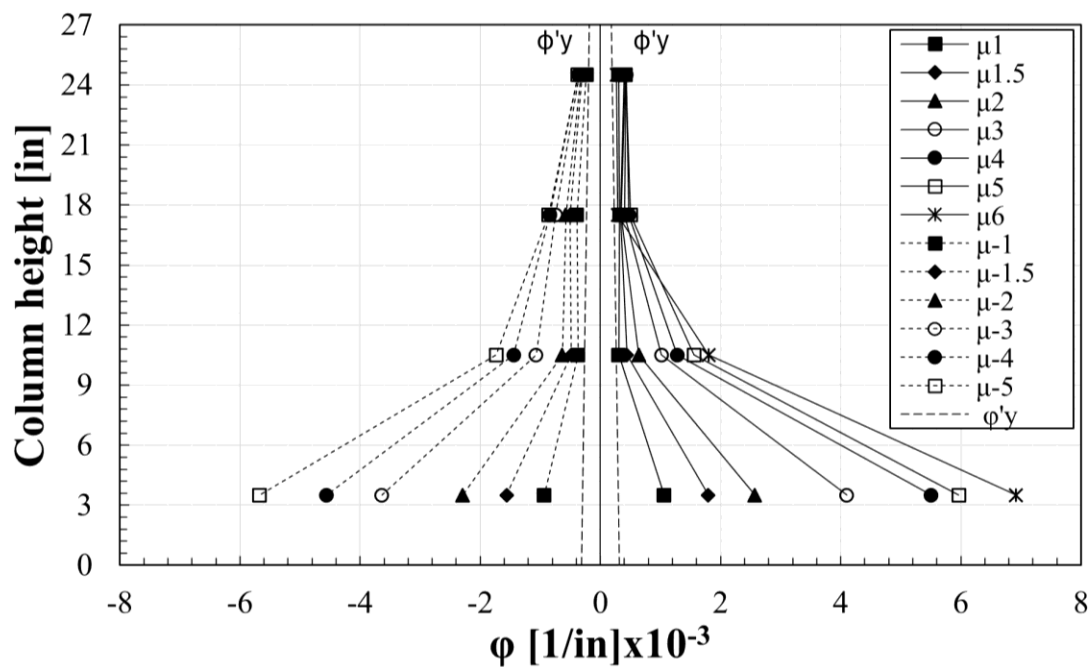


Figure 5-39 High-level curvature profile of Column 1: MA_RT_2in

5.5. Experimental response COL2: QST RT 2IN

5.5.1. Summary

The second test was performed at room temperature on an 18-inch-diameter column reinforced with 10 #6 QST ASTM A706 grade 80 bars. The specimen reached a maximum force of 53 kips and exhibited a 20% strength loss during the third pushing cycle of μ_5 and the first pulling cycle of μ_6 , due to the fracture of the extreme bars. The column was loaded

until the first pushing cycle of μ_6 , at which point it lost approximately 70% of its strength.

A summary of the column parameters is presented in Table 5-11.

Table 5-11 Summary of column parameters

Column parameters	
Longitudinal Steel:	10 #6 ASTM A706 Gr.80 ($\rho_l=1.74\%$) QST
Transverse Steel:	#3 ASTM A706 Gr.80 @ 2" ($\rho_t=1.23\%$)
Longitudinal Steel Nominal Yield Strength:	86.4 ksi
Transverse Steel Nominal Yield Strength:	91.2 ksi
Concrete Strength:	7 ksi
Axial Load:	123 kips
Axial Load Ratio:	6.92%
First Yield Displacement:	0.58 in
F'y	39 kip
Nominal Moment Capacity:	282 kip-ft
Equivalent Yield Displacement:	0.8 in
Maximum Lateral Force:	53 kips
Displacement at 20% Strength loss:	4 in
Failure Mode:	Fracture of buckled bar

5.5.2. Test observations

Starting from the cycle at $\frac{1}{2} F'y$, hairline cracks formed along the plastic hinge and continued expanding throughout the test. In the first cycle of μ_3 , the concrete began to flake. Although no visible buckling was observed for the extreme top bar (2T), significant concrete crushing was seen along the bar at the first pulling cycle of μ_5 , which could be an indication of the initiation of buckling. This bar (2T) fractured in the third pushing cycle of μ_5 . The extreme bottom bar (2B) buckled at approximately the third pushing cycle of μ_5 and fractured in the first pulling cycle of μ_6 . The bars 1T and 3T buckled in the first pulling cycle of μ_6 and fractured in the following pushing cycle. The test was finalized after the fracture of bars 1T and 3T, as the column exhibited significant strength degradation. Figure 5-40 to Figure 5-53 illustrate the progression of the column's damage during the test. Table 5-12

presents the summary of the damage observations from the test of the second column specimen.

Table 5-12 Column 2 damage progression summary

Damage observations	
Initiation of cracking:	1/2 F'y
Core Concrete Crushing:	$\mu 3^{+3}$
Longitudinal Bar Buckling Top:	None visible
Longitudinal Bar Buckling Bottom:	$\mu 5^{-3}$
Longitudinal Bar Fracture Top:	$\mu 5^{-3}$
Longitudinal Bar Fracture Bottom:	$\mu 6^{+1}$
20% Strength loss (-):	$\mu 5^{-3}$
20% Strength loss (+):	$\mu 6^{+1}$
(+) Pulling, (-) Pushing	



Figure 5-40 First cycle of ¼ F'y (pulling)

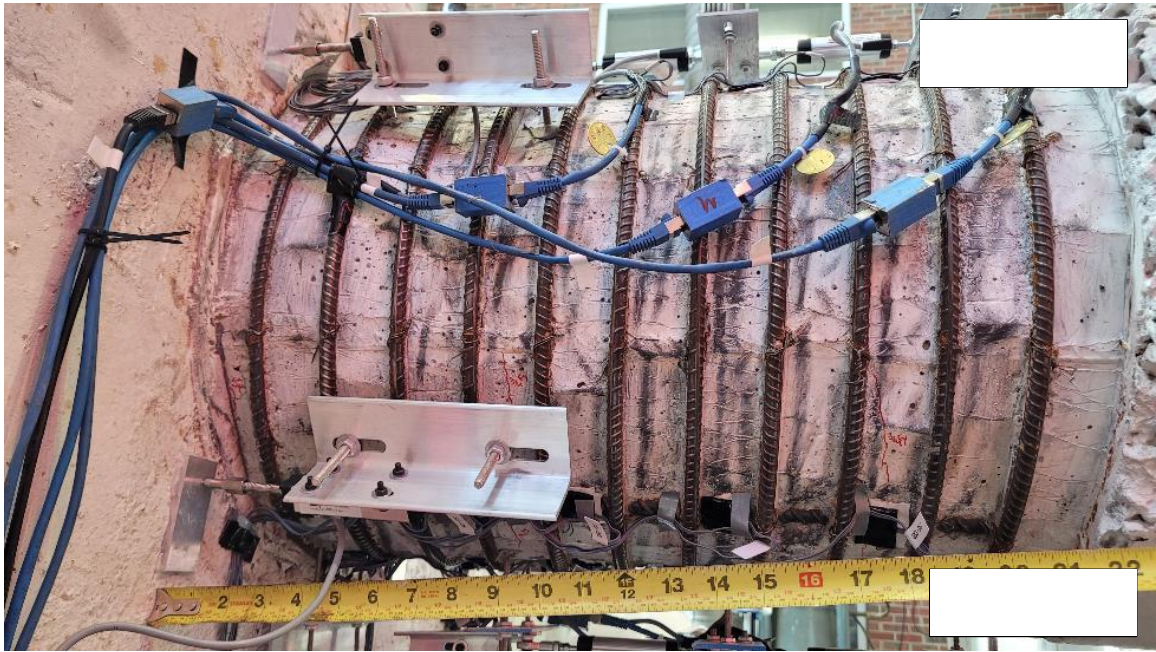


Figure 5-41 First cycle of $3/4 F_y$ (pulling): Hairline cracks formed along the plastic hinge

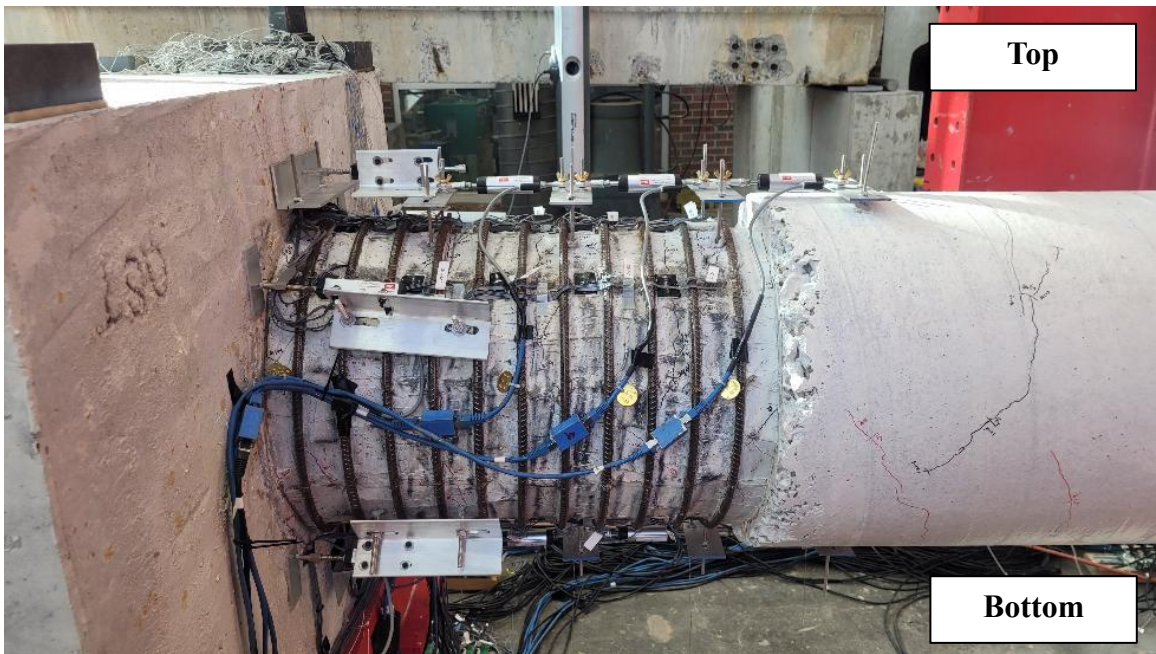


Figure 5-42 First cycle μ_1 (pushing): Shear cracks formed along the plastic hinge. Some perimetral cracks were observed at the base of the column and over the plastic hinge.

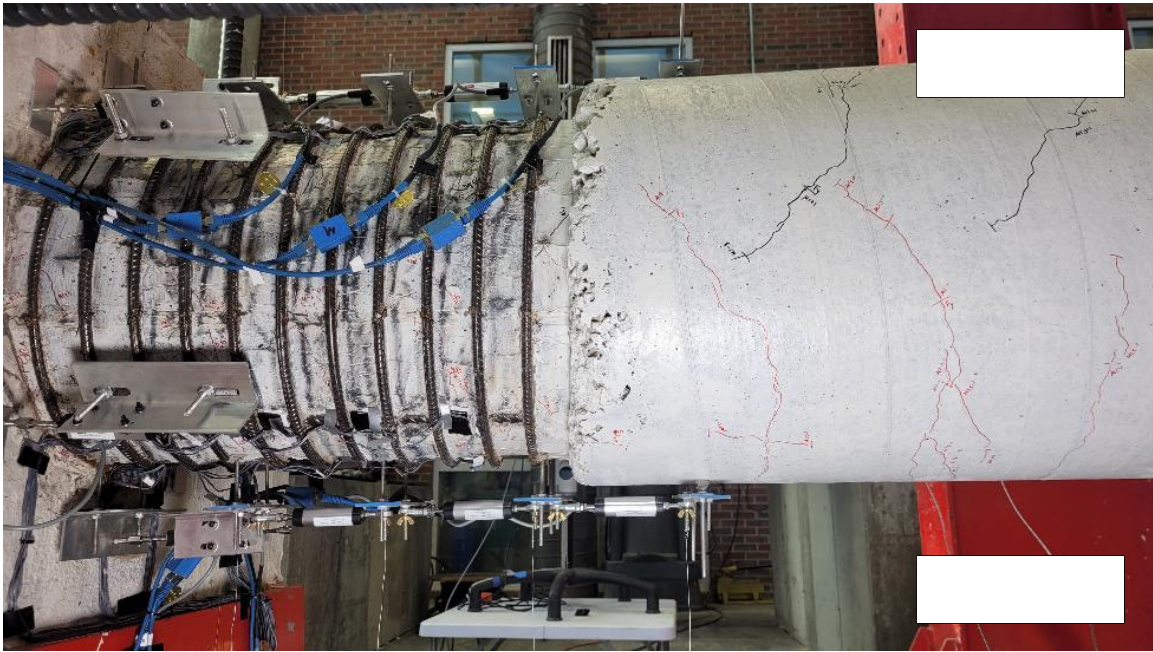


Figure 5-43 First cycle $\mu 1.5$ (pulling): Cracks elongated and expanded. Some cracks formed parallel to the reinforcing steel over the plastic hinge.

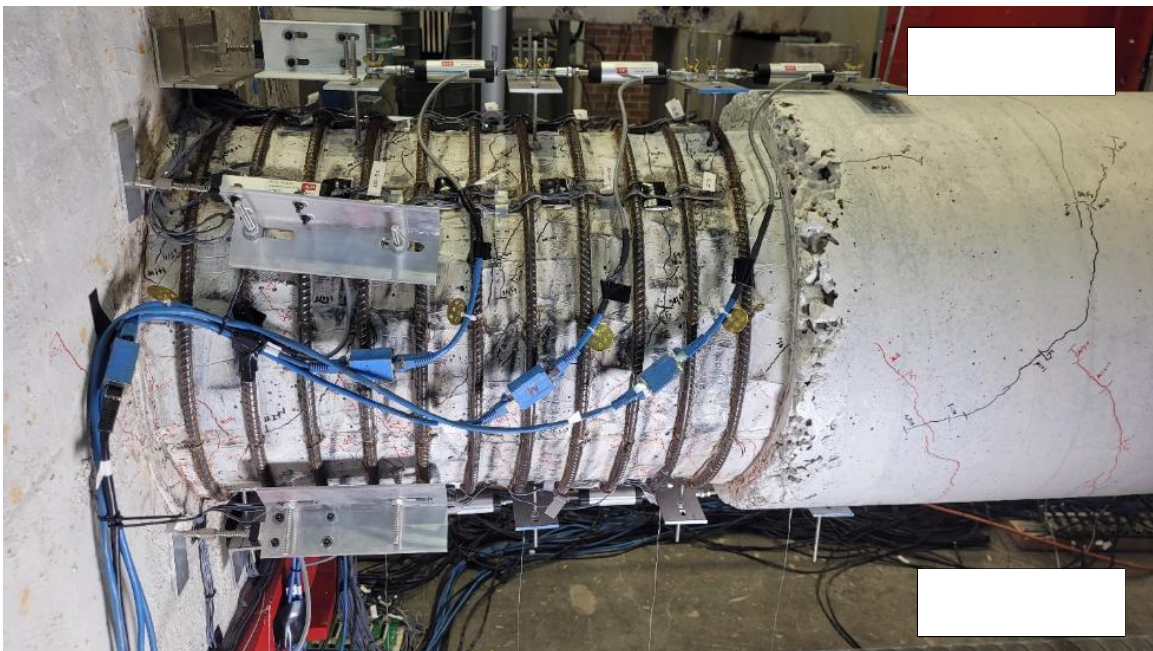


Figure 5-44 Third cycle $\mu 2$ (pulling): Cracks at the base of the column expanded, and cracks formed at the footing.

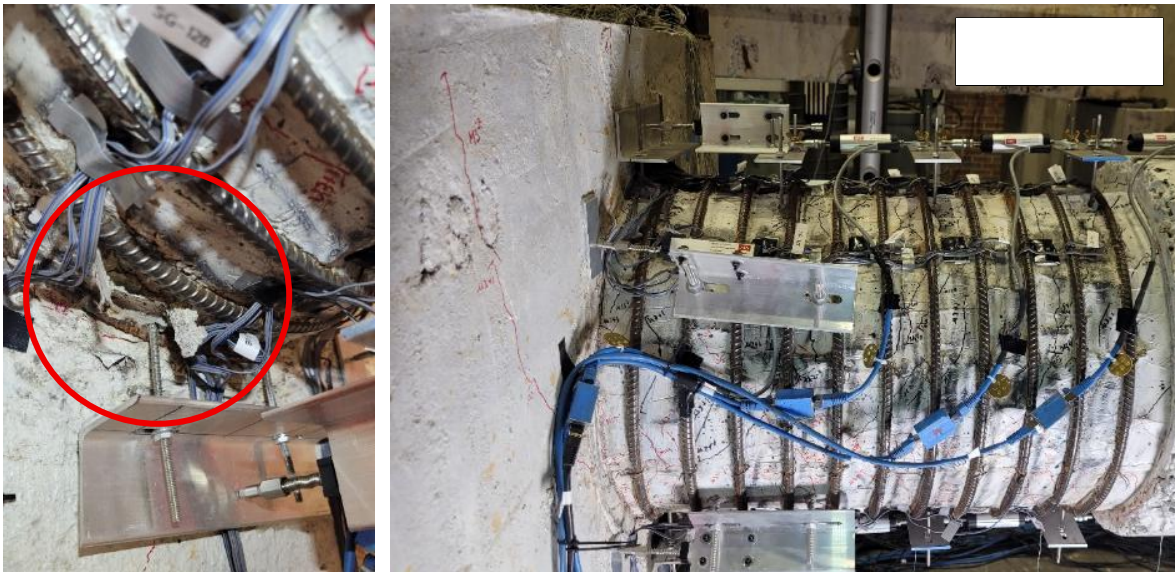


Figure 5-45 Third cycle μ_3 (pushing): Extension of cracks and concrete crushing.



Figure 5-46 First cycle of μ_4 (pulling).



Figure 5-47 Third cycle of μ_4 (pulling): Cracks are widening at the base of the footing.



Figure 5-48 Third cycle of μ_5 (pulling and pushing): Concrete crushing around the extreme bar 2T, followed by its fracture in the subsequent pushing cycle.



Figure 5-49 First cycle μ_6 (pulling): buckling of top bar 3T, and fracture of bottom extreme bar 2B



Figure 5-50 First cycle of μ_6 (pushing): fracture of top bars 1T and 3T



Figure 5-51 Damage at the top of the column along the plastic hinge: Fracture of bars 1T, 2T, and 3T, and concrete core crushing.



Figure 5-52 Damage at the bottom of the column: Fracture of bar 2B, concrete core crushing.



Figure 5-53 Experimental extent of plasticity.

5.5.3. Force-displacement response

The hysteretic response of the column is shown in Figure 5-54, where each ductility level and bar fracture is indicated. As with the previous specimen, the column exhibited fractures in four bars. The first bar fractured during the last pushing cycle of μ_5 , which corresponded to the extreme top bar (2T). At this point, a 20% drop in strength was observed in the pushing direction (-). Similarly, a 20% drop in strength was observed in the pulling direction (+) after the fracture of the extreme bottom bar (2B), which occurred at the first pulling cycle of μ_6 . The two bars adjacent to the extreme top bar (1T and 3T) fracture at the first pushing cycle of μ_6 (one complete cycle earlier than the MA specimen), at which point a drop in strength of about 70% of the maximum force was observed. These observations are also shown in Figure 5-55, which presents the envelope of the force-displacement response of the column, with each graph corresponding to the envelope for the first, second, and third cycles at each ductility level.

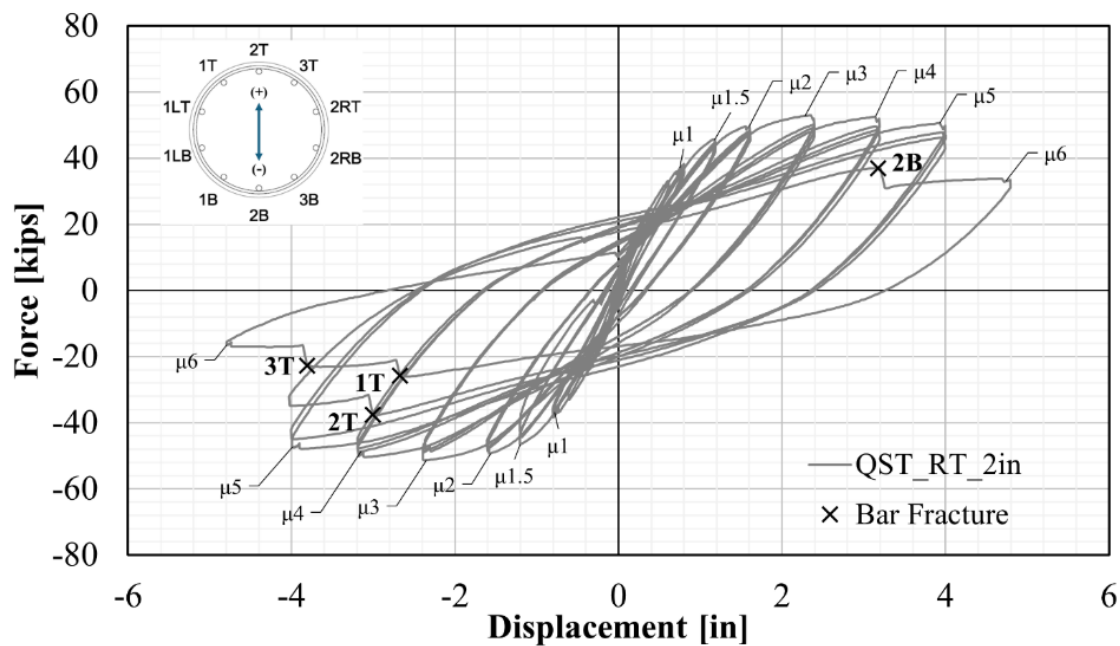


Figure 5-54 Hysteretic response of Column 2: QST_RT_2in

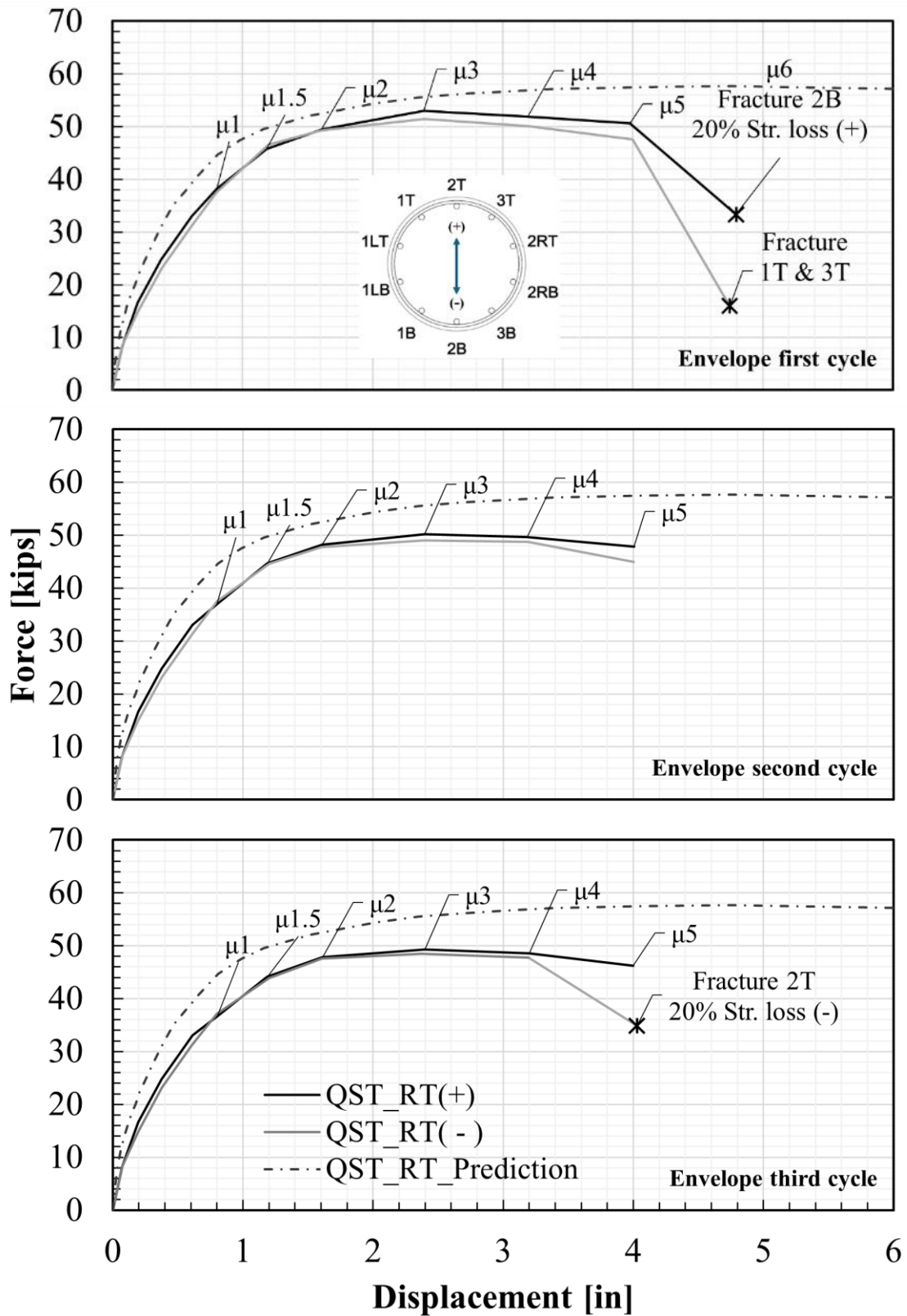


Figure 5-55 Envelope of the Force-displacement response of the RC column specimen #2.

5.5.4. Curvature Profile

The low-level and high-level curvature profiles were generated for Column2:QST_RT_2in, respectively. The predicted first yield curvature is also presented, assuming a linear distribution from the base of the column to the top of the column. In both low-level and high-level profiles, the distribution of curvatures was very symmetric, with the high-level curvature showing slightly higher values in the pulling direction. At the low-level curvature profile, the maximum curvature values seemed to be between the first and the second gages near the base of the column, which corresponds to the location that is between 3.5 in and 10.5 in above the footing (Figure 5-56). However, for the high-level profiles, the higher values of curvature shifted towards the base of the column (Figure 5-57). The curvature profiles for μ_6 in the pushing and pulling directions were not included in the high-level curvature profile due to damage to the column that affected the instrument support at the base and, therefore, the accuracy of the data.

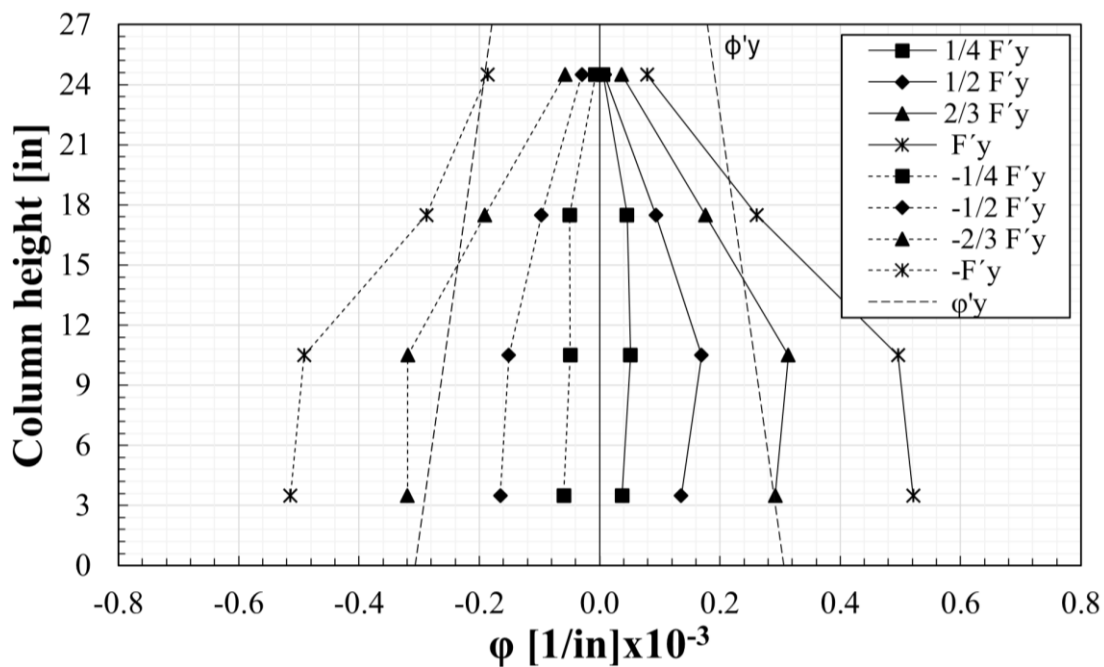


Figure 5-56 Low-level curvature profile of Column 2: QST_RT_2in

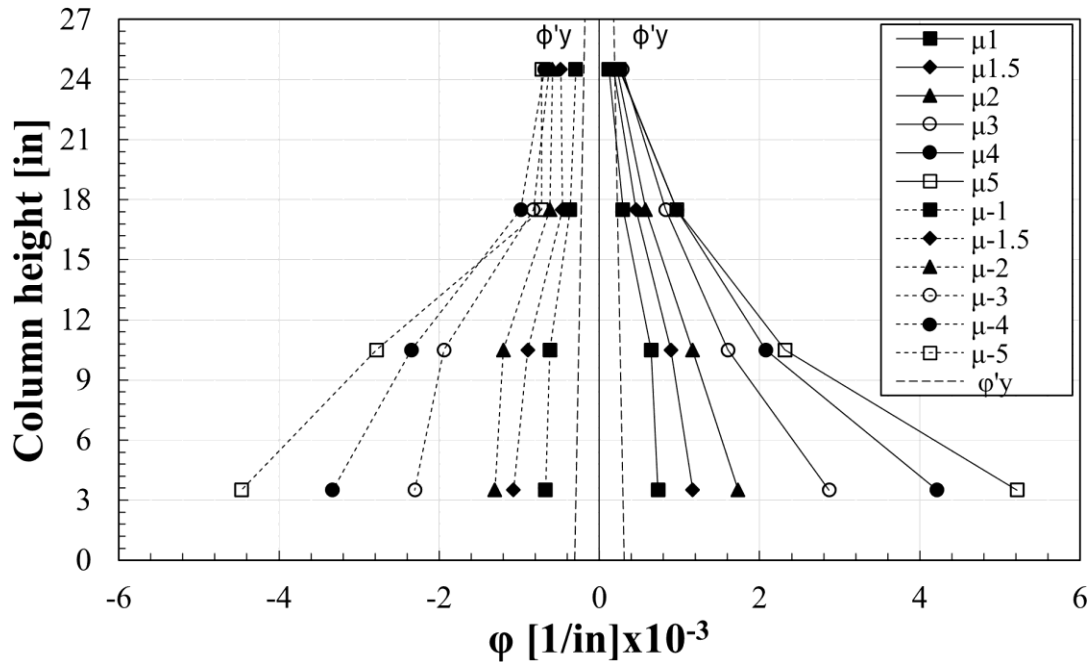


Figure 5-57 High-level curvature profile of Column 2: QST_RT_2in

5.6. Experimental response COL3: MA LT 2IN

5.6.1. Summary

The third test was performed at low temperature on an 18-inch-diameter column reinforced with 10 #6 MA ASTM A706 grade 80 bars. The column was conditioned for about 37 hours at a set temperature of -40°F, reaching -36°F at the reinforcing steel. The column maintained an average temperature of -37°F during the test. During the test, the specimen reached a maximum force of 59 kips and exhibited a 20% strength loss during the first pushing cycle of μ_5 and the second pulling cycle of μ_6 , due to the fracture of the extreme bars. The column was loaded to the third μ_6 pulling cycle, at which point it lost approximately 50% of its strength. A summary of the column parameters is presented in Table 5-13. Figure 5-58 presents the temperature record during conditioning and the test.

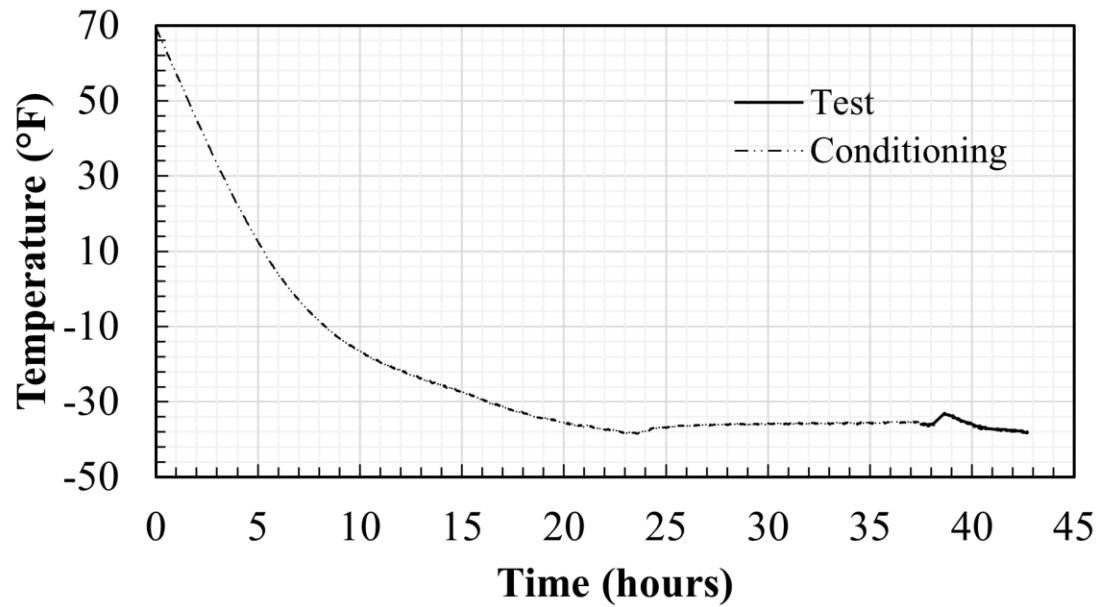


Figure 5-58 Temperature record of column 3:MA_LT_2in during the conditioning time and testing.

Table 5-13 Summary of column parameters

Column parameters	
Longitudinal Steel:	10 #6 ASTM A706 Gr.80 ($\rho_l=1.74\%$) MA
Transverse Steel:	#3 ASTM A706 Gr.80 @ 2" ($\rho_t=1.23\%$)
Longitudinal Steel Nominal Yield Strength:	84.5 ksi
Longitudinal Steel Nominal Yield Strength LT:	90.5 ksi
Transverse Steel Nominal Yield Strength RT:	91.2 ksi
Transverse Steel Nominal Yield Strength LT:	97.6 ksi
Concrete Strength RT:	6.70 ksi
Concrete Strength LT:	9.50 ksi
Axial Load:	123 ksi
Axial Load Ratio:	7.70%
First Yield Displacement:	0.53 in
F'y:	39.2 kip
Nominal Moment Capacity:	268 kip-ft
Equivalent Yield Displacement:	0.7 in
Maximum Lateral Force:	59 kips
Displacement at 20% Strength loss:	4 in
Failure Mode:	Fracture of buckled bar

5.6.2. Test observations

Hairline cracks started forming at $\frac{3}{4} F'y$ along the plastic hinge. The concrete began to crush during the first cycle of μ_4 , at a higher displacement than that of the MA column

pair tested at room temperature. During the first pulling cycle of $\mu 5$, the column exhibited bar buckling of the top extreme bar (2T), which fractured during the subsequent pushing cycle. An additional top bar (1T) fractured during the first pushing cycle of $\mu 6$. At this same cycle, a spiral yielding along the bottom extreme bar (2B) was observed, which could be an indicator of bar buckling. The bottom extreme bars (2B) fractured in the following pulling cycle of $\mu 6$. A total of three bars fractured in this test: 2 top bars and 1 bottom bar. The test was finalized after the fracture of the extreme bottom bar, as the column exhibited significant strength degradation. Figure 5-59 to Figure 5-70 present the progression of the column's damage during the test. Table 5-14 presents the summary of the damage observations from the test of the third column specimen.

Table 5-14 Column 3 damage progression summary

Damage observations	
Initiation of cracking:	$3/4 F'y$
Core Concrete Crushing:	$\mu 4^{+1}$
Longitudinal Bar Buckling Top:	$\mu 5^{+1}$
Longitudinal Bar Buckling Bottom:	$\mu 6^{-1}$
Longitudinal Bar Fracture Top:	$\mu 5^{-1}$
Longitudinal Bar Fracture Bottom:	$\mu 6^{+2}$
20% Strength loss (-):	$\mu 5^{-1}$
20% Strength loss (+):	$\mu 6^{+2}$
(+) Pulling, (-) Pushing	



Figure 5-59 Experimental setup of the low-temperature quasistatic lateral cyclic column test



Figure 5-60 First cycle of μ_1 (pushing)

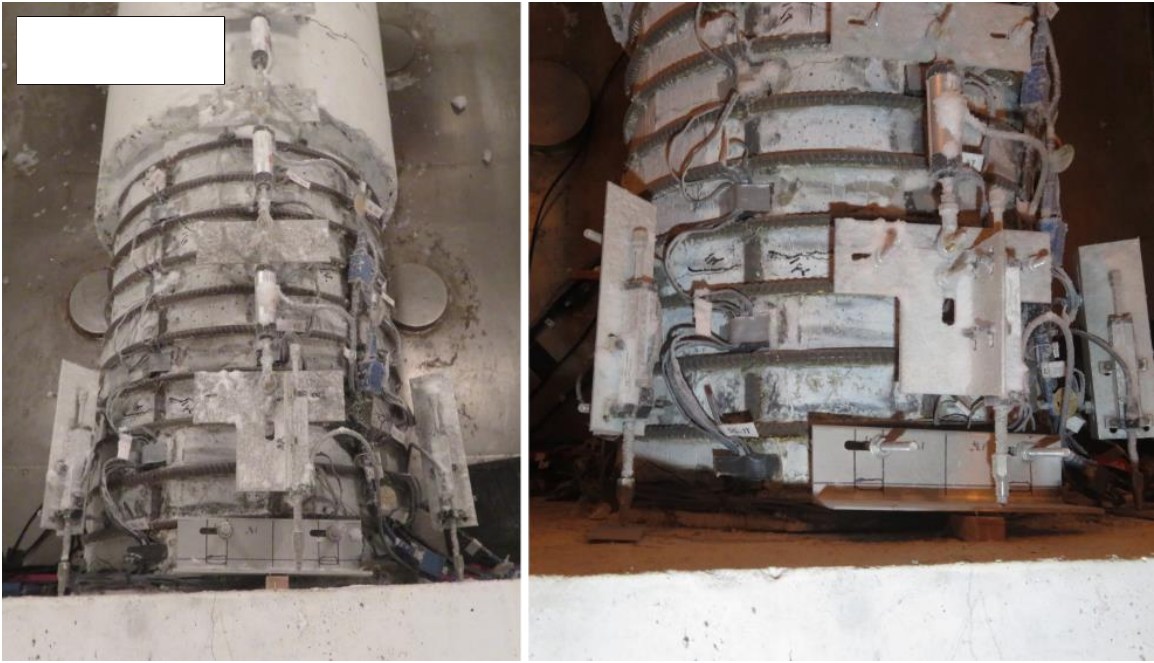


Figure 5-61 First cycle of μ_2 (pushing). Perimetral cracks expand along the plastic hinge



Figure 5-62 First cycle of μ_3 (pushing). Some hairline shear cracks were observed along the column, and expansion of cracks at the base of the columns



Figure 5-63 First cycle of μ_4 (pulling). Initiation of concrete crushing



Figure 5-64 First cycle of μ_5 (pushing). Concrete damage around the longitudinal bars, damage observed in the footing, and thickening of cracks along the plastic hinge length



Figure 5-65 First cycle of $\mu 5$: bar 2T buckled at the first pulling cycle and fractured in the subsequent pushing cycle



Figure 5-66 First pushing cycle of $\mu 6$, fracture of top bar T1



Figure 5-67 Second cycle of $\mu 6$ (pulling). Fracture the bottom extreme bar 2B



Figure 5-68 Damage at the top of the column along the plastic hinge: Fracture of bars 1T and 2T, and concrete core crushing.

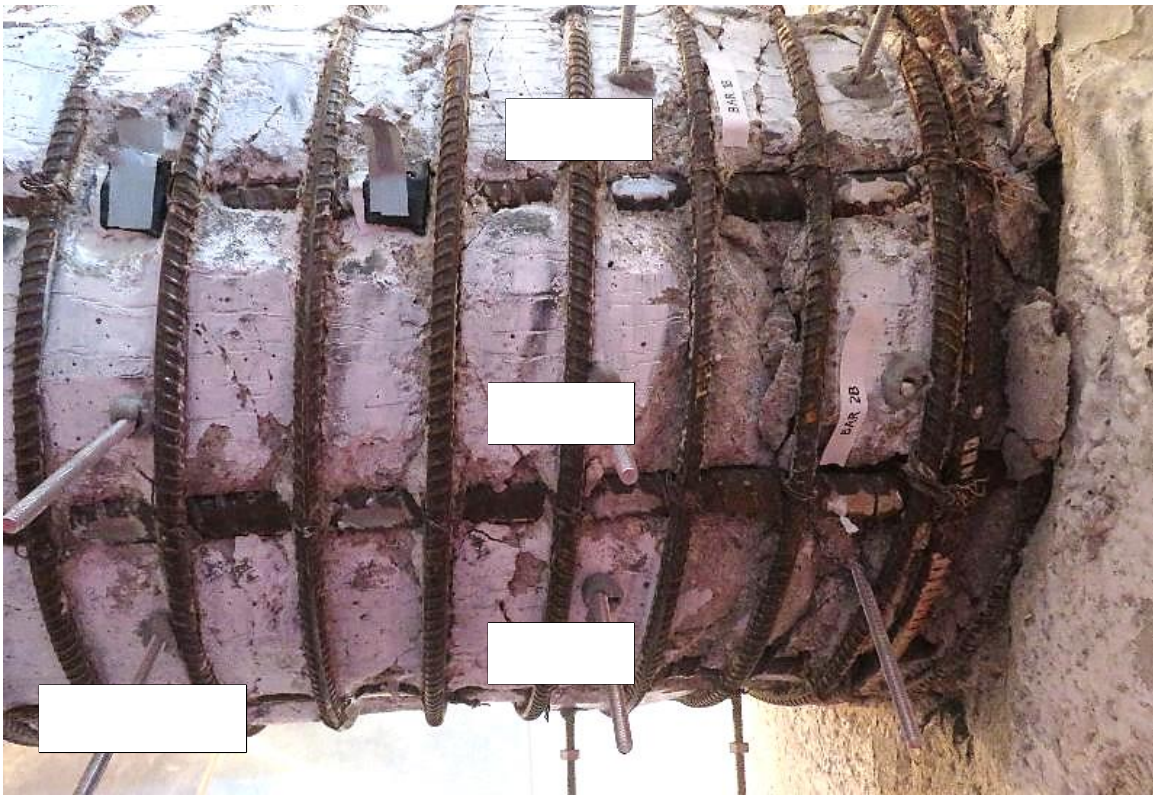


Figure 5-69. Damage at the bottom of the column: Fracture of the extreme bottom bar 2B, concrete core crushing, and some footing damage.

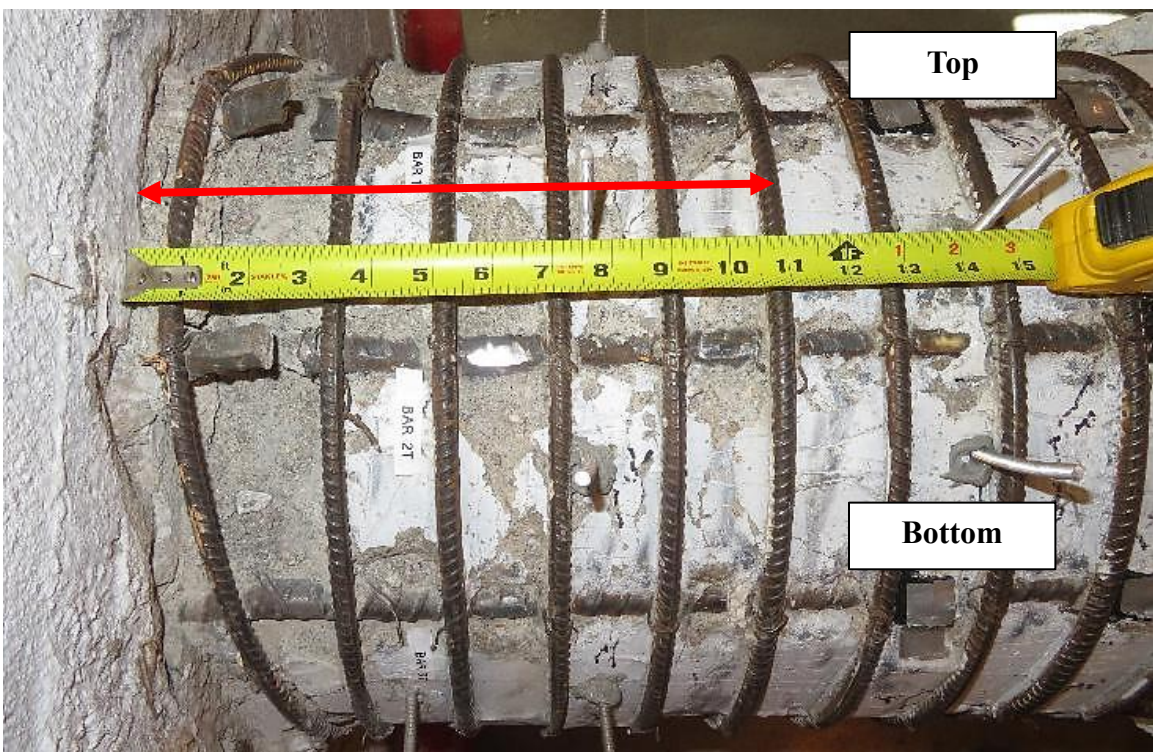


Figure 5-70. Experimental extent of plasticity.

5.6.3. Force-displacement response

The hysteretic response of the column is shown in Figure 5-71, indicating each ductility level and bar fractures. The column presented fracture on three bars. The extreme top bar (2T) fractured first during the first pushing cycle of μ_5 , right after visible buckling appeared in the previous pulling cycle. At this point, a 20% drop in strength was observed in the pushing direction (-). Compared with the MA pair column tested at room temperature, the fractured bar appeared more brittle. First, the room-temperature specimen underwent three complete loading cycles, from bar buckling to bar fracture. In contrast, the low-temperature column exhibited bar fracture immediately after visible buckling. In addition, the fracture of the top extreme bar in the low-temperature specimen occurred at a displacement far from the target displacement of the cycle (-4in), where the column was still pulled beyond its zero position, as shown in Figure 5-72.

During the first pushing cycle of μ_6 , the column exhibited fracture of the top bar T1 and spiral yielding around the bottom extreme bar 2B, which could indicate buckling. The test was run until the third pulling cycle of μ_6 , after the fracture of the extreme bottom bar, at which point the column strength degraded by around 50%. These observations are also shown in Figure 5-73, which presents the envelope of the force-displacement response of the column, with each graph corresponding to the envelope for the first, second, and third cycles at each ductility level.

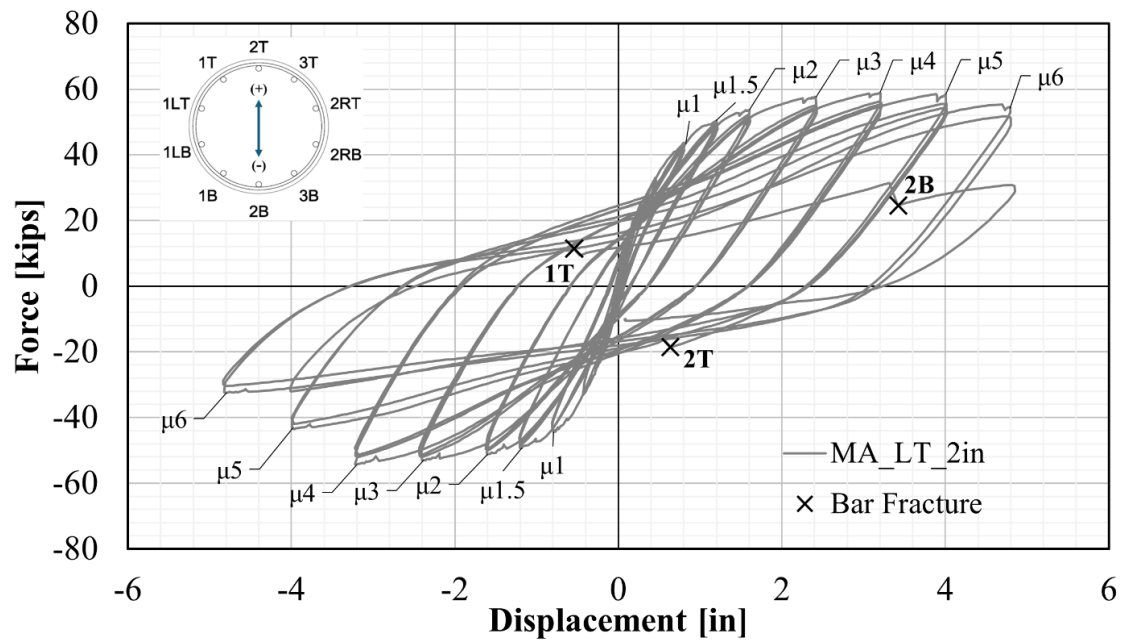


Figure 5-71 Hysteretic response of Column 3: MA_LT_2in

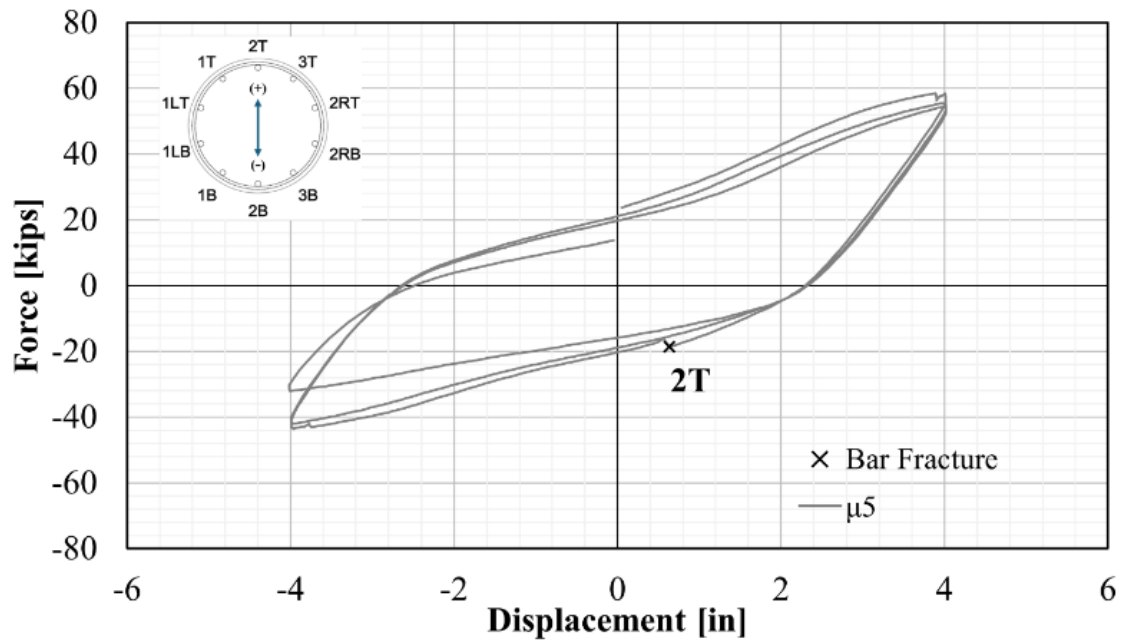


Figure 5-72 Hysteretic response of Column 3: MA_LT_2in at μ_5

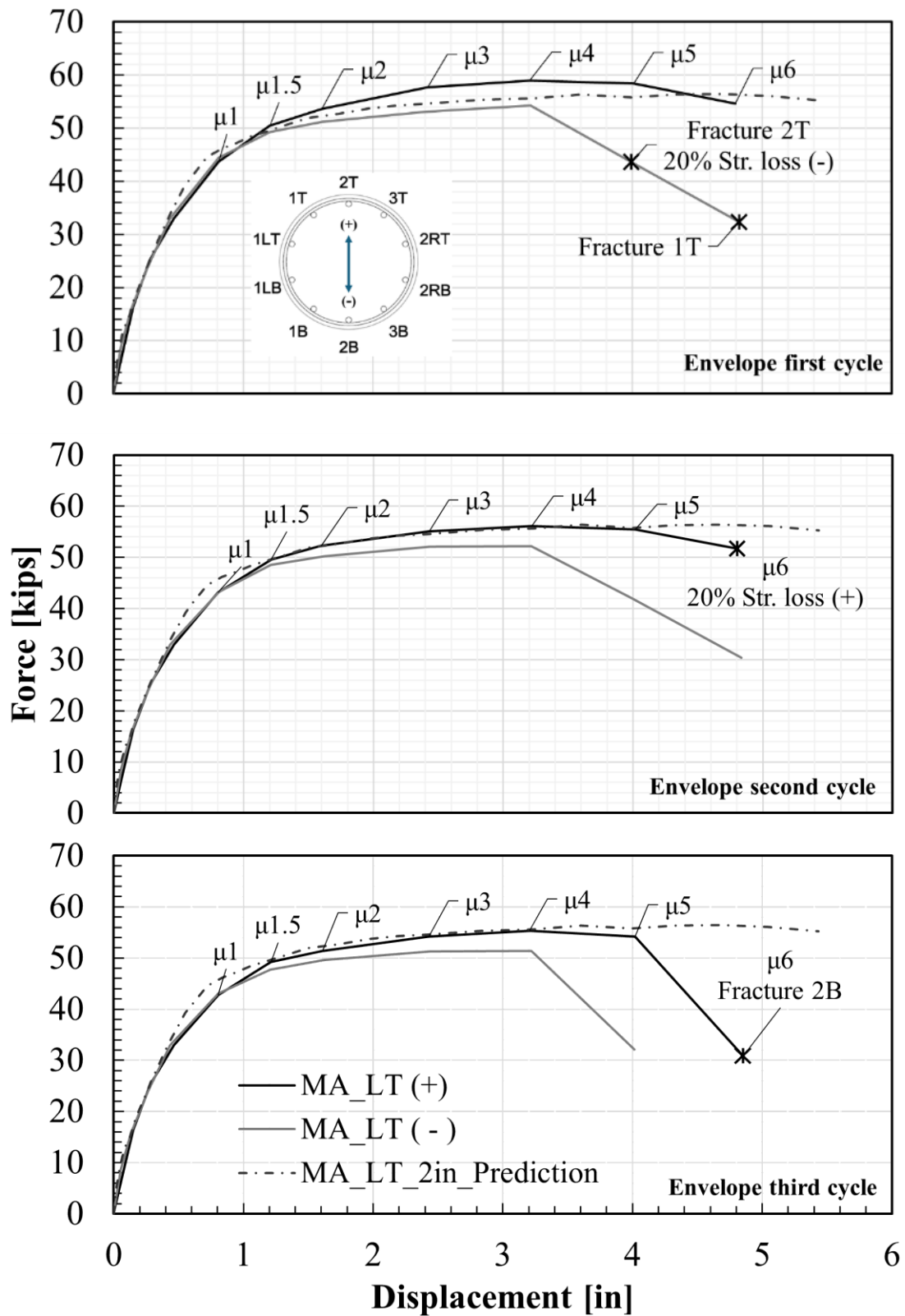


Figure 5-73 Envelope of the Force-displacement response of the RC column specimen #3

5.6.4. Curvature Profile

Figure 5-74 and Figure 5-75 present the low-level and high-level curvature profiles generated for Column3:MA_LT_2in, respectively. The predicted first-yield curvature is also provided, assuming a linear distribution from the base of the column to the top of the column. The distribution of curvatures in the low-level curvature profile differs between the column's pushing and pulling directions. Figure 5-74 shows that the higher curvature values are concentrated in the second gage for the pulling direction (solid lines) and in the base of the columns for the pushing directions (dashed lines). However, these concentrations shifted as the column was subjected to higher displacements. This is observed in the high-level curvature profile as the distribution of curvatures became more symmetric, with the greater values concentrated at the base of the column for all ductilities (Figure 5-75). The curvature distributions for μ_5 and μ_6 were not included in the high-level profile due to an inconsistency in the data caused by the instrument support due to column damage.

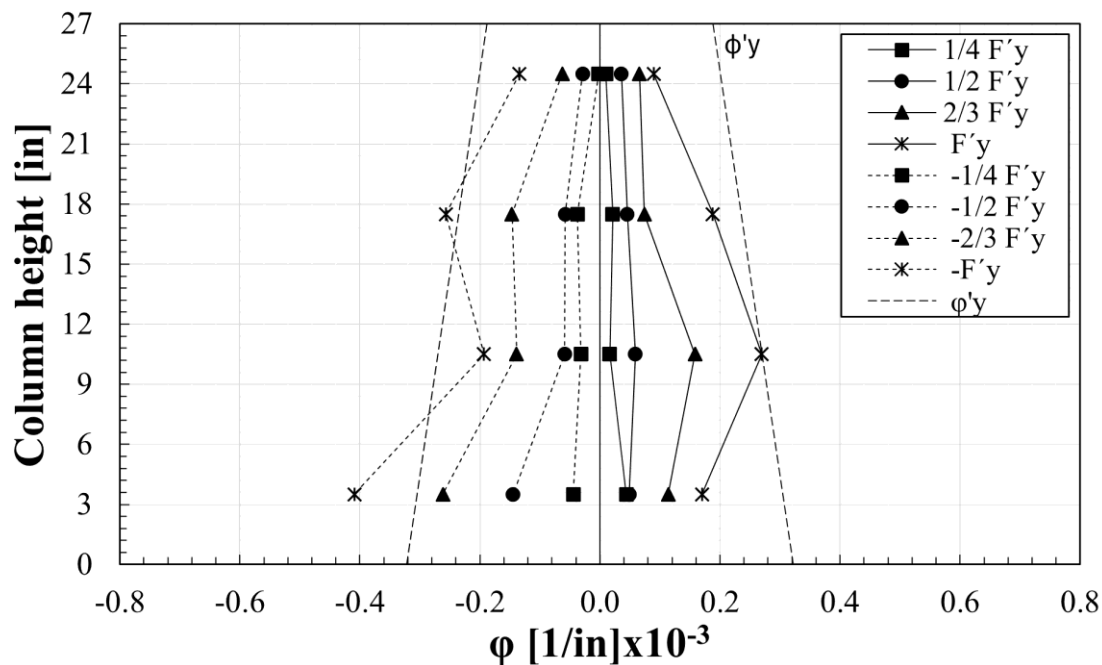


Figure 5-74 Low-level curvature profile of Column 3: MA_LT_2in.

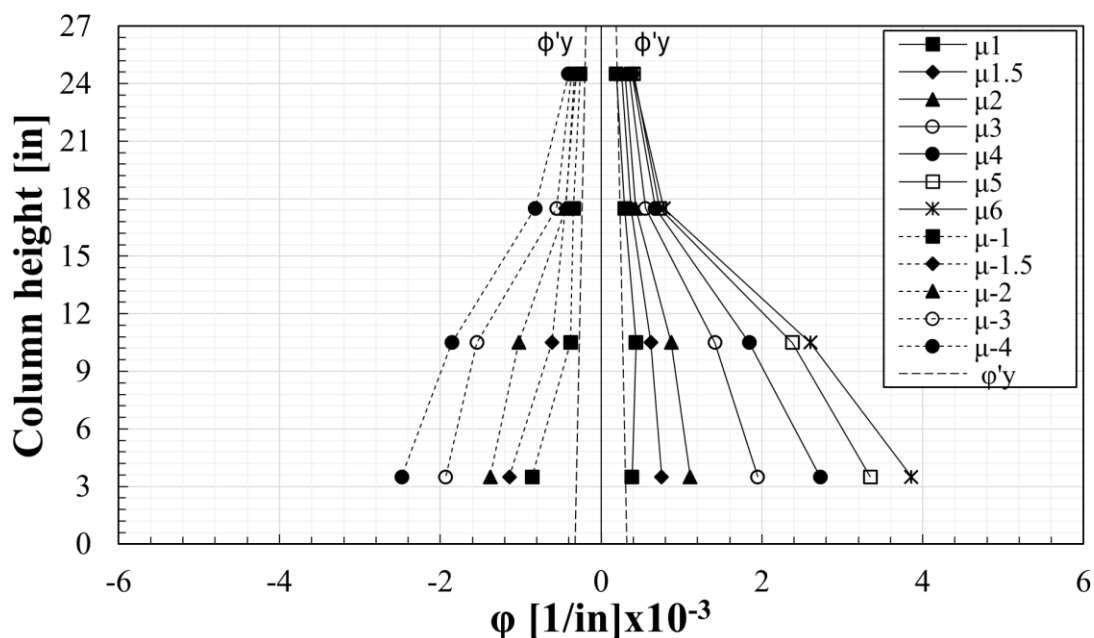


Figure 5-75 High-level curvature profile of Column 3: MA_LT_2in.

5.7. Experimental response COL4: QST LT 2IN

5.7.1. Summary

The fourth test was performed at low temperature on an 18-inch-diameter column reinforced with 10 #6 QST ASTM A706 grade 80 bars. The column was conditioned for approximately 37 hours at a set temperature of -40°F, reaching -38.6°F at the reinforcing steel. The column maintained an average temperature of -37°F during the test. Figure 5-76 presents the temperature record during the conditioning time and the test.

During the test, the specimen reached a maximum force of 59 kips and exhibited a 20% strength loss during the first pushing cycle of μ5, due to the fracture of the top extreme bar. In the pulling direction, the 20% strength loss occurred in the second pulling cycle of μ5, before the extreme bottom bar fracture. The column was loaded until the third pushing cycle of μ5, at which point it lost approximately 60% of its strength. A summary of the column parameters is presented in Table 5-15.

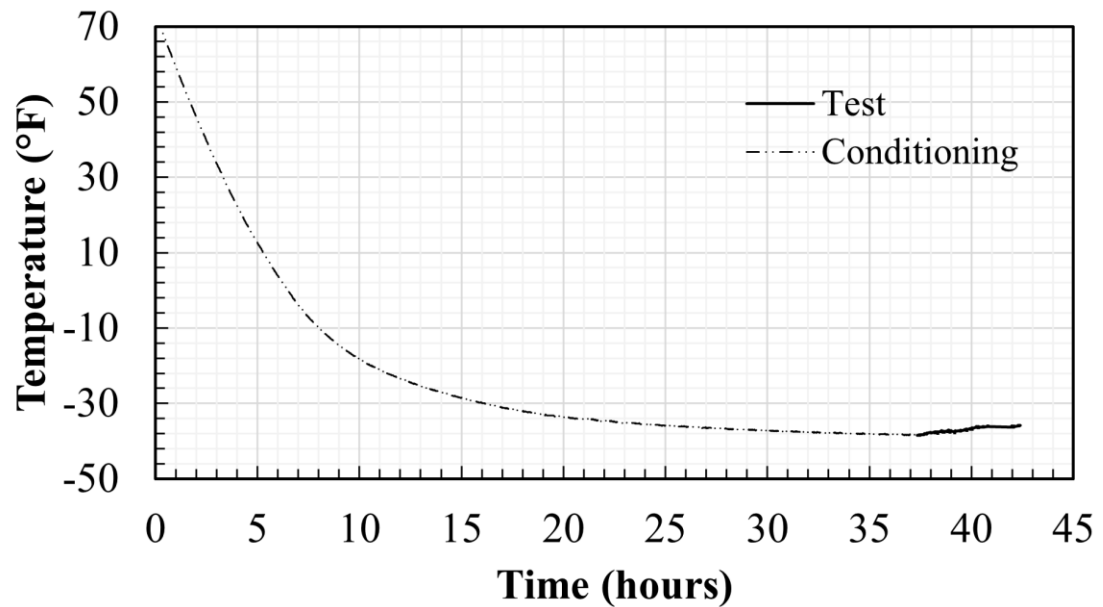


Figure 5-76 Temperature record of column 4: QST_LT_2in during the conditioning time and testing.

Table 5-15 Summary of column parameters

Column parameters	
Longitudinal Steel:	10 #6 ASTM A706 Gr.80 ($\rho_l=1.74\%$) QST
Transverse Steel:	#3 ASTM A706 Gr.80 @ 2" ($\rho_t=1.23\%$)
Longitudinal Steel Nominal Yield Strength:	86.4 ksi
Longitudinal Steel Nominal Yield Strength LT:	92.3 ksi
Transverse Steel Nominal Yield Strength RT:	91.2 ksi
Transverse Steel Nominal Yield Strength LT:	97.9 ksi
Concrete Strength RT:	7.14 ksi
Concrete Strength LT:	9.34 ksi
Axial Load:	123 ksi
Axial Load Ratio:	6.77%
First Yield Displacement:	0.57 in
F'y	40.3 kip
Nominal Moment Capacity:	287 kip-ft
Equivalent Yield Displacement:	0.7 in
Maximum Lateral Force:	58 kips
Displacement at 20% Strength loss:	4 in
Failure Mode:	Fracture of buckled bar

5.7.2. Test observations

The first hairline cracks along the plastic hinge appeared at F'y and continued expanding throughout the test. Concrete crushing was observed in the first pulling cycle of $\mu 4$. In the third pushing cycle of $\mu 4$, the spiral around the extreme top bar started yielding, which could indicate initiation of buckling for bar 2T. The buckling for this bar was visible in the first pulling cycle of $\mu 5$, followed by its fracture in the subsequent pushing cycle. An additional top bar (3T) fractured in the first pushing cycle of $\mu 5$. At this point, the column exhibited a 20% strength degradation in the pushing direction (+). In the second pushing cycle of $\mu 5$, the adjacent bar to the top extreme bar (1T) fractured, leading to a three-bar fracture at the top of the column by the end of the second cycle of $\mu 5$. The column was loaded to the third pulling cycle at $\mu 5$, at which point bar fracture of the bottom extreme bar (2B) occurred, resulting in approximately 40% strength degradation in the pulling direction (-). The visible spread of plasticity extended to about 10 ¼ inches from the column base. Figure 5-77 to Figure 5-87 present the progression of the column's damage during the test. Table 5-16 presents the summary of the damage observations from the test of the fourth column specimen.

Table 5-16 Column 4 damage progression summary

Damage observations	
Initiation of cracking:	F'y
Core Concrete Crushing:	$\mu 4^{+1}$
Longitudinal Bar Buckling Top:	$\mu 4^{+3}$
Longitudinal Bar Buckling Bottom:	$\mu 5^{-1}$
Longitudinal Bar Fracture Top:	$\mu 5^{+1}$
Longitudinal Bar Fracture Bottom:	$\mu 5^{-3}$
20% Strength loss (+):	$\mu 5^{+1}$
20% Strength loss (-):	$\mu 5^{-2}$
(+) Pulling, (-) Pushing	



Figure 5-77 First cycle of μ_1 (pushing)

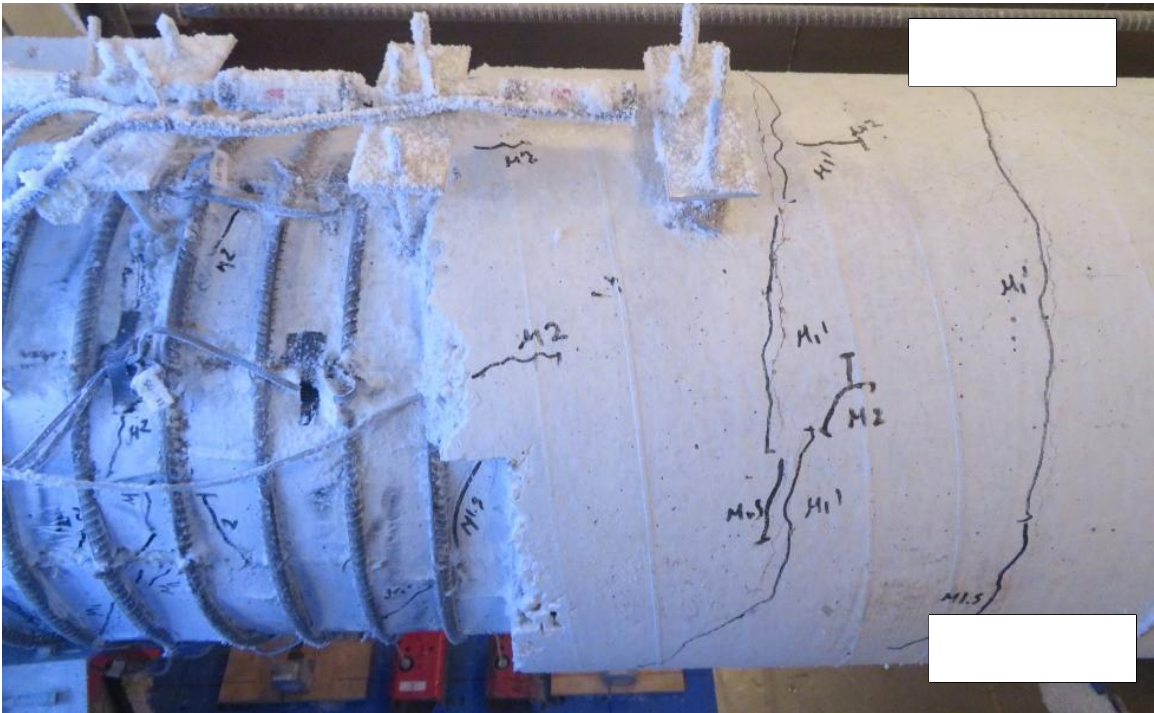


Figure 5-78 First cycle of μ_2 (pushing)



Figure 5-79 First cycle of μ_3 (pushing) visible shear cracks and extension of existing cracks



Figure 5-80 Third cycle of μ_3 (pushing) widening of the perimetral cracks near the base of the column



Figure 5-81 First cycle of μ_4 (pulling): Concrete crushing



Figure 5-82 Spiral yielding in the third pulling cycle of μ_4 and visible bar buckling in the first pulling cycle of μ_5 – Top extreme bar 2B



Figure 5-83 Fracture of the top bars 2T and 1T in the first pushing cycle of $\mu 5$



Figure 5-84 Bar buckling of the extreme bottom bar 2B at the first pushing cycle of $\mu 5$ and fracture of the bar 2B at the third pulling cycle $\mu 5$



Figure 5-85 Damage at the top of the column along the plastic hinge: Fracture of bars 1T, 2T, and 3T, and concrete core crushing

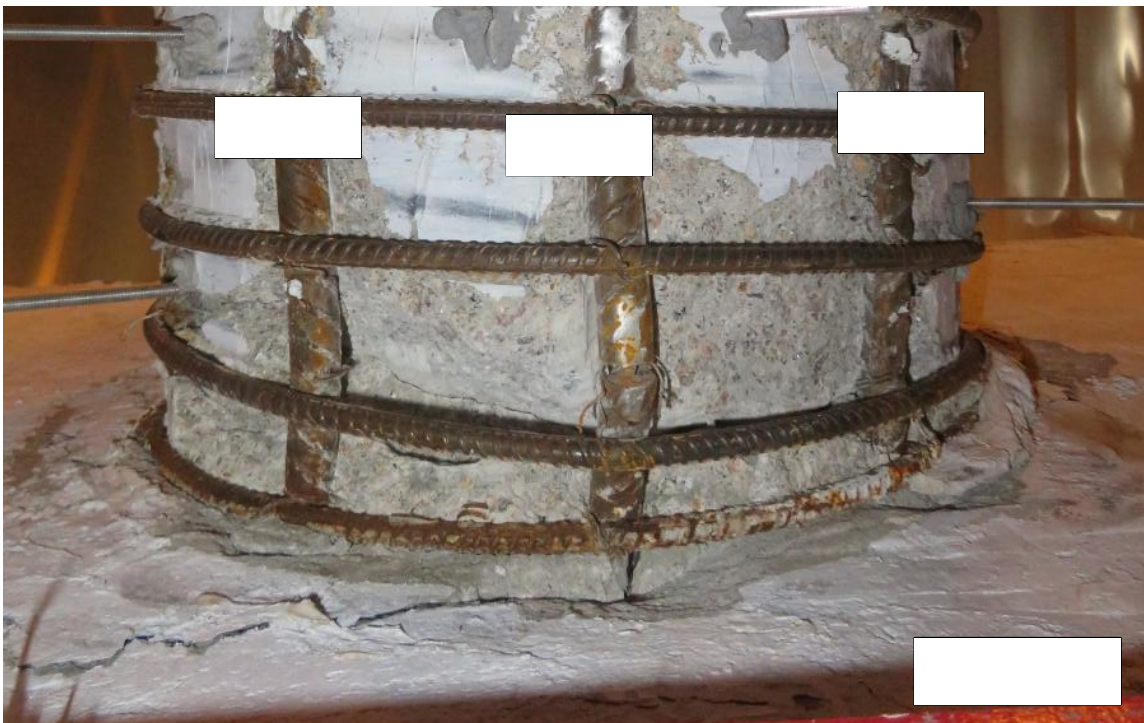


Figure 5-86 Damage at the bottom of the column along the plastic hinge: Fracture of bar 2B, and concrete core crushing



Figure 5-87 Experimental extent of plasticity

5.7.3. Force-displacement response

Figure 5-88 presents the hysteretic response of the column at all applied ductility levels and indications of bar fracture. The specimen presented fractures in four bars. Two bars fractured in the first pushing cycle of μ_5 , which corresponded to the extreme top bar and its adjacent bar (2T and 3T). At this point, a 40% drop in strength was observed in the pushing direction (-) due to the fracture of both bars. A third bar fractured during the second pushing cycle of μ_5 , corresponding to top bar 1T, and caused a 20% drop in strength in the pulling direction (+). Lastly, the bottom extreme bar (2B) fracture occurred in the third cycle of μ_5 . The test was finalized at this point, as the strength in the pushing direction had decreased by 60% from its maximum. These observations are also evident in the force-displacement envelopes of the columns shown in Figure 5-89.

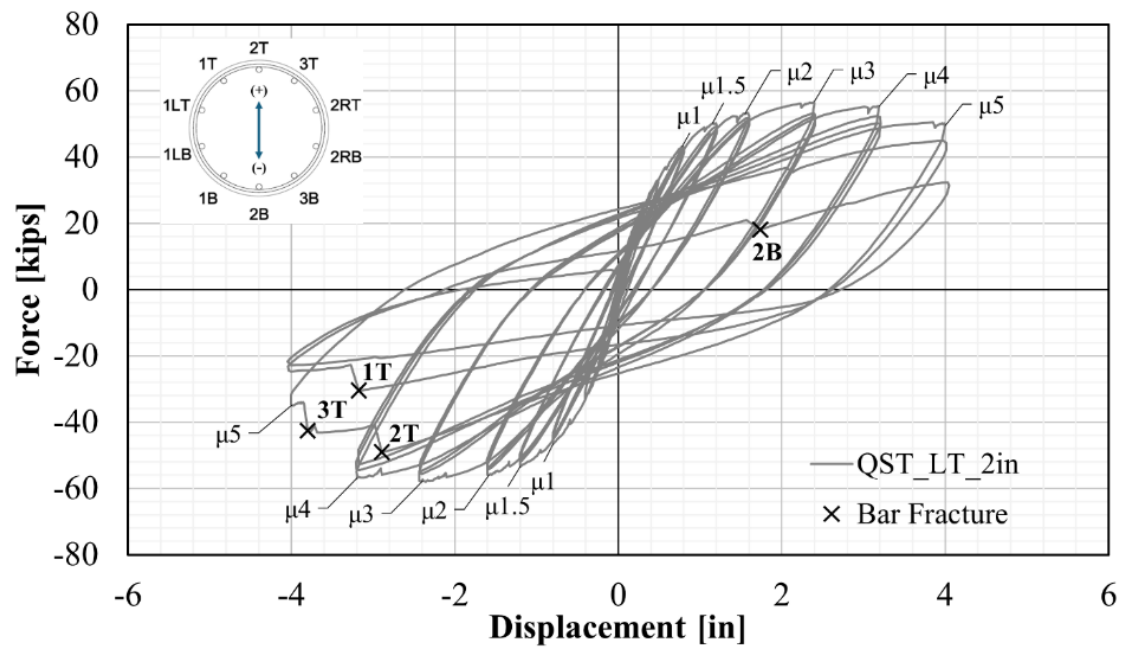


Figure 5-88 Hysteretic response of Column 4: QST_LT_2in

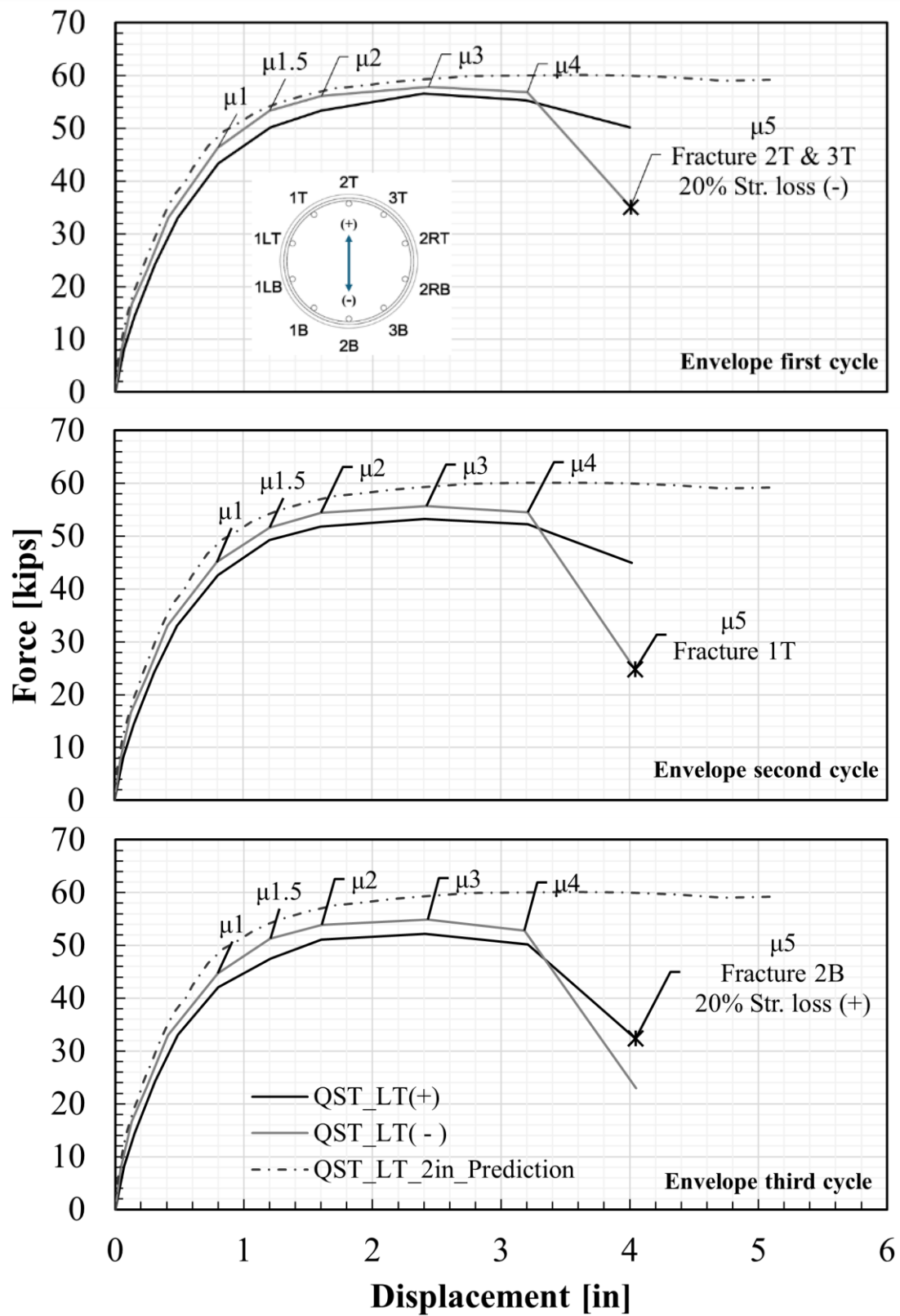


Figure 5-89 Envelope of the Force-displacement response of the RC column specimen #4

5.7.4. Curvature Profile

The low-level and high-level curvature profiles were generated for Column4:QST_LT_2in. Figure 5-90 show the low-level curvature distribution, where a symmetric distribution is observed between the pushing and pulling cycles. It is also noted that, in the low-level distribution, higher curvature values were obtained in the second gage, corresponding to a position of 10.5 inches above the column's footing. The concentration of curvature on the second gage was maintained until μ_2 , as seen in the high-level curvature profile presented in Figure 5-91. From μ_3 , higher curvature values were observed near the base of the column. In the pushing direction, the curvatures showed the maximum values between the first and second gages, indicating a greater spread of plasticity in this direction. While in the case of the pulling direction, the maximum curvature values were concentrated more at the base of the column, indicating more concentration of damage at the top of the column. The curvature distribution corresponding to μ_5 was not presented for either loading direction as the damage of the column led to damage on the support of the instrument, resulting in unreliable results for this ductility level.

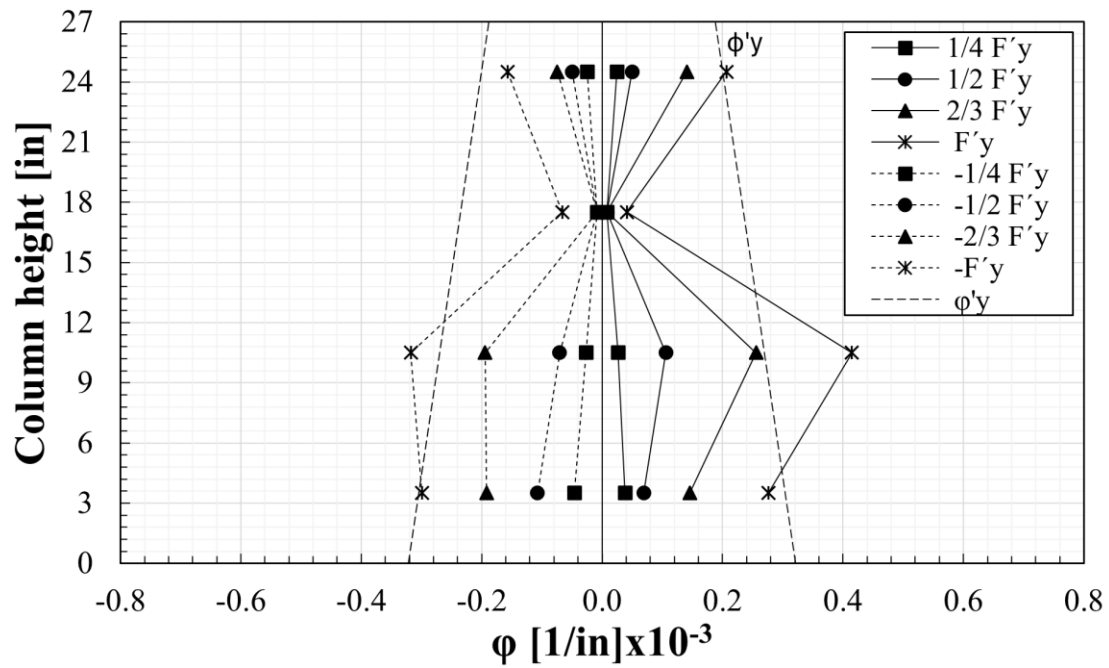


Figure 5-90 Low-level curvature profile of Column 4: QST_LT_2in

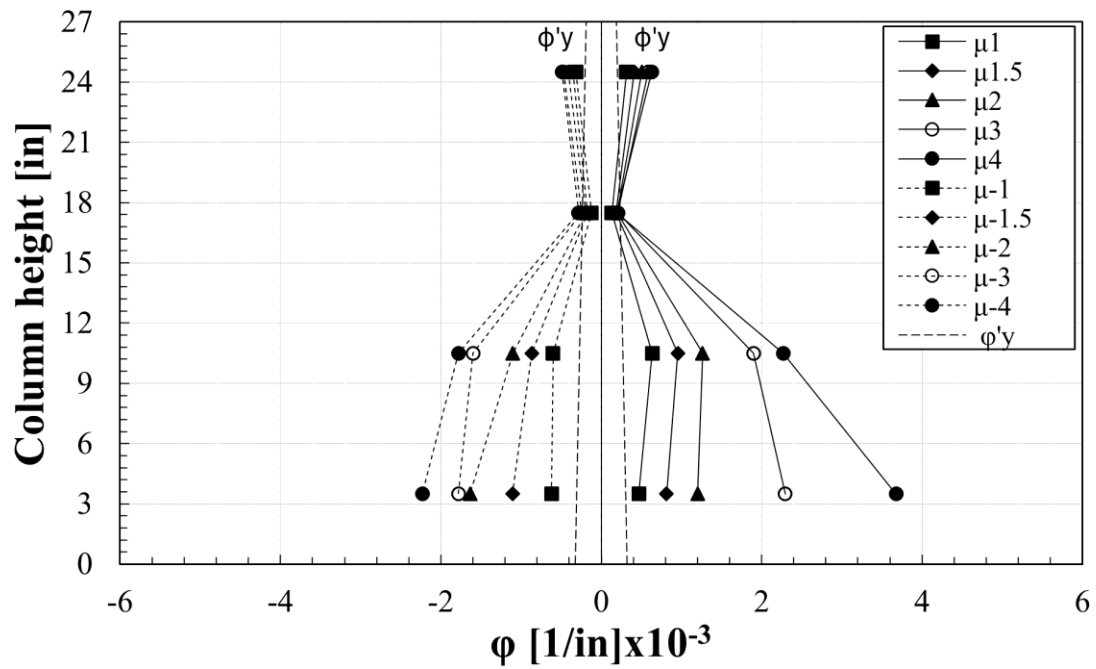


Figure 5-91 High-level curvature profile of Column 4: QST_LT_2in

5.8. Experimental response COL5: MA LT 4IN

5.8.1. Summary

Column five corresponded to the last pair of columns that were constructed using a larger spiral spacing of 4 inches. This specimen was constructed using 10 #6 MA ASTM A706 grade 80 bars as longitudinal steel and tested at low temperatures. The column was conditioned for approximately 37 hours, achieving a temperature of -38°F on the reinforcing steel. During the test, the column maintained an average temperature of -36.4°F on the reinforcing steel. The records of the conditioning and test temperatures are shown in Figure 5-92.

Column 5 reached a maximum strength of 57 kips and presented a 20% strength degradation in the second pushing cycle and third pulling cycle of $\mu 5$, caused by the fracture of the longitudinal extreme bars. The column was loaded until the third pushing cycle of $\mu 5$, at which point a 60% drop in strength was observed. A summary of the column parameters for the fifth specimen is presented in Table 5-17.

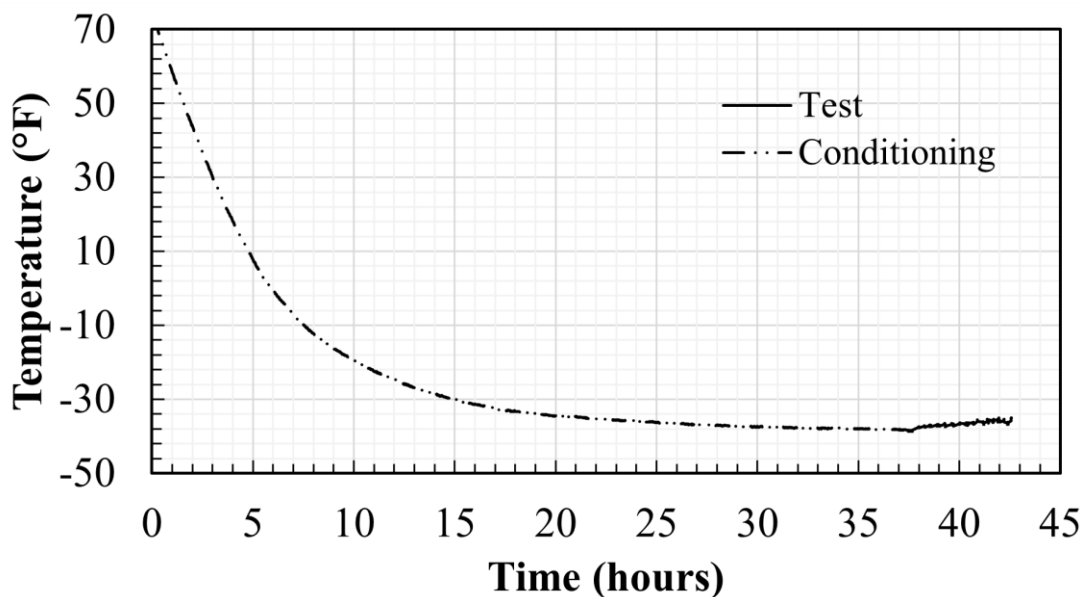


Figure 5-92 Temperature record of column 5:MA_LT_4in during the conditioning time and testing

Table 5-17 Summary of column parameters

Column parameters	
Longitudinal Steel:	10 #6 ASTM A706 Gr.80 ($\rho_l=1.74\%$) MA
Transverse Steel:	#3 ASTM A706 Gr.80 @ 2" ($\rho_t=1.23\%$)
Longitudinal Steel Nominal Yield Strength RT:	84.5 ksi
Longitudinal Steel Nominal Yield Strength LT:	90.5 ksi
Transverse Steel Nominal Yield Strength RT:	91.2 ksi
Transverse Steel Nominal Yield Strength LT:	97.6 ksi
Concrete Strength_RT:	4.50 ksi
Concrete Strength_LT:	7.80 ksi
Axial Load:	123 ksi
Axial Load Ratio:	10.7%
First Yield Displacement:	0.55 in
F'y	37.6 kip
Nominal Moment Capacity:	203 kip-ft
Equivalent Yield Displacement:	0.7 in
Maximum Lateral Force:	57 kips
Displacement at 20% Strength loss:	4 in
Failure Mode:	Fracture of buckled bar

5.8.2. Test observations

The initiation of cracking was observed at $\frac{3}{4} F'y$ along the plastic hinge. The hairline cracks continued to expand along the test. During the third μ_3 pulling cycle, concrete crushing was observed at the top of the column. In the first pulling cycle of μ_4 , significant concrete crushing was observed around the longitudinal extreme bar 2T; this was followed by visible bar buckling in the first pulling cycle of μ_5 . The first bar fracture in the second pushing cycle of μ_5 , with two cycles of load between visible bar buckling and the fracture. The fracture of the top extreme bar 2T led to a 20% drop in strength in the pushing direction. The bottom extreme bar (2B) fractured in the third pulling cycle of μ_5 ; no visible buckling was seen for this bar. An additional top bar (3T), adjacent to the extreme top bar, fractured during the third pushing cycle of μ_5 , resulting in a 60% strength degradation and ending the test. A total of three bars fractured in this test: 2 top bars and 1 bottom bar. The fractured bars at the top of the column presented a brittle failure surface, consistent with the type of

failures observed in the material-level evaluation, where the broken surface presented a "stair step" shape (Figure 5-103).

The photographic records indicated that a fourth bar fractured at the top of the column (Figure 5-102). This fracture was not observed during the test and was not visible in the hysteretic response, so it likely occurred after the test was finalized, when the column was being unloaded. Figure 5-93 to Figure 5-106 show the damage progression of the column. Table 5-18 presents the summary of the damage observations from the test of the third column specimen.

Table 5-18 Column 5 damage progression summary

Damage observations	
Initiation of cracking:	3/4 f_y
Core Concrete Crushing:	$\mu 3^{+3}$
Longitudinal Bar Buckling Top:	$\mu 5^{+1}$
Longitudinal Bar Buckling Bottom :	None visible
Longitudinal Bar Fracture Top:	$\mu 5^{-2}$
Longitudinal Bar Fracture Bottom:	$\mu 5^{+3}$
20% Strength loss (+):	$\mu 5^{-2}$
20% Strength loss (-):	$\mu 5^{+3}$
(+) Pulling, (-) Pushing	

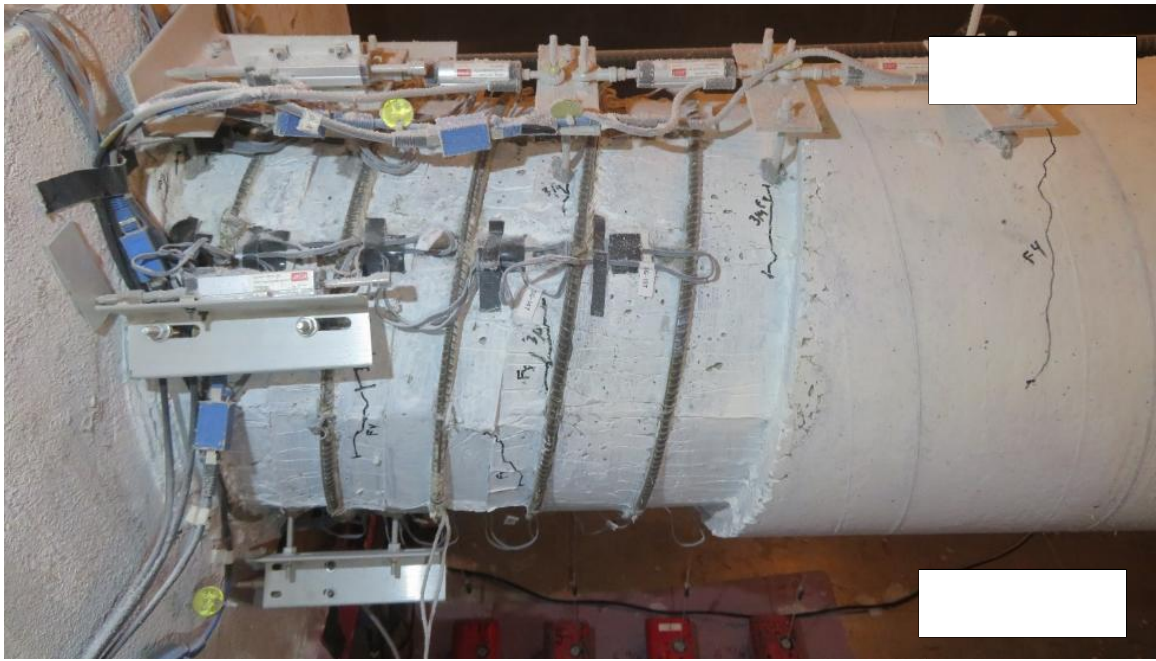


Figure 5-93 F_y (pulling): Hairline cracks formed along the plastic hinge in the first cycle of $3/4 F_y$ and continued expanding



Figure 5-94 First cycle of μ_1 (pushing): Extension of cracks



Figure 5-95 Third cycle of μ_2 (pushing)

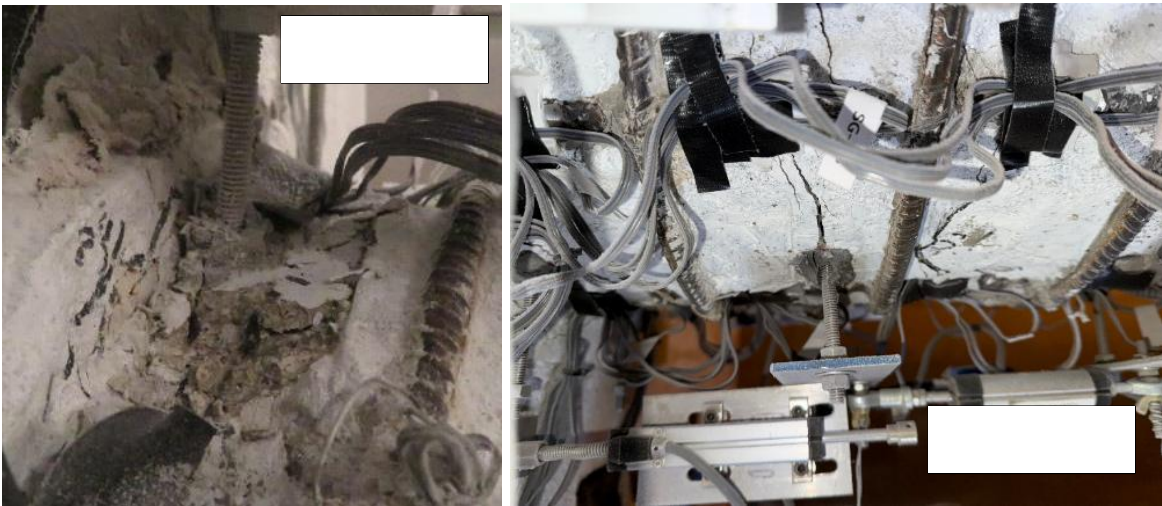


Figure 5-96 Third pulling cycle of μ_3 : Concrete crushing at the top of the column and cracks widening at the bottom



Figure 5-97 Third pulling cycle of $\mu 4$: Significant concrete crushing around the longitudinal top extreme bar, 2T



Figure 5-98 First pulling cycle of $\mu 5$: Visible buckling of top extreme bar, 2T



Figure 5-99 Second pushing cycle of $\mu 5$: Bar fracture of top extreme bar, 2T



Figure 5-100 Third pulling cycle of $\mu 5$: Visible buckling of adjacent bar to top bar, 3T



Figure 5-101 Third pushing cycle of $\mu 5$: Bar fracture of bottom bar, 3B



Figure 5-102 Damage at the top of the column along the plastic hinge: Fracture of bars 1T, 2T, and 3T, and concrete core crushing



Figure 5-103 Brittle fracture surface of top bars (1T, 2T, and 3T)

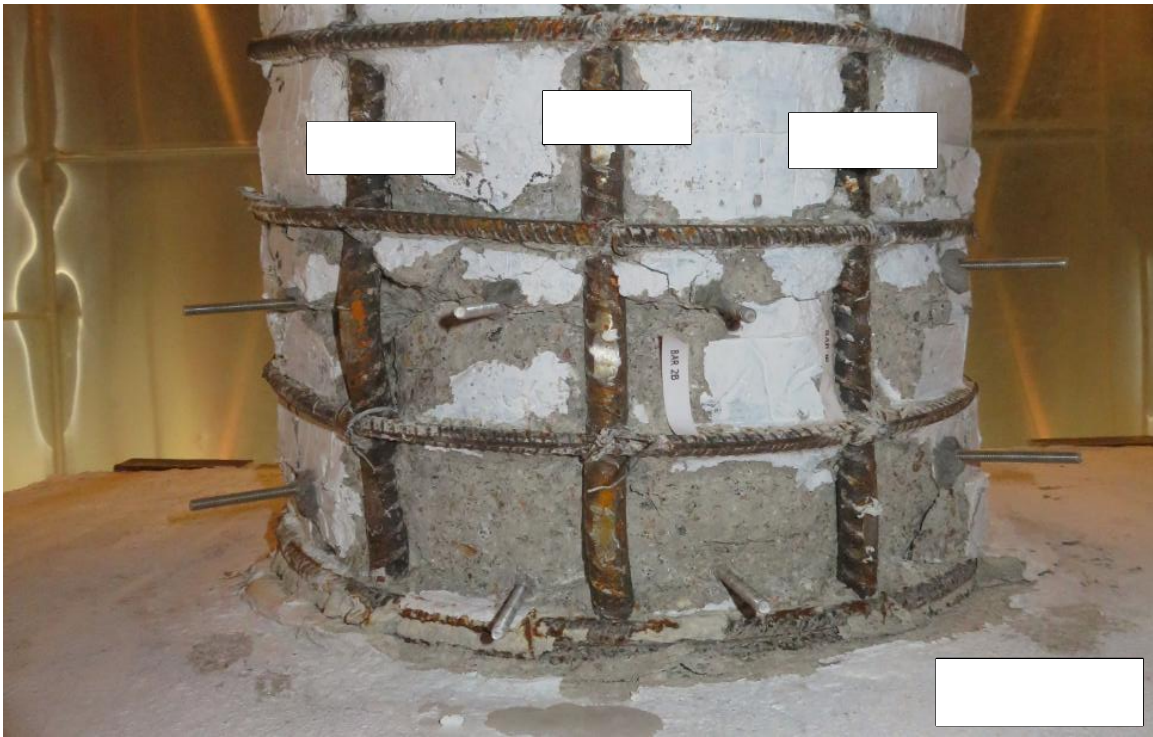


Figure 5-104 Damage at the bottom of the column along the plastic hinge: Fracture of bar 2B and concrete core crushing



Figure 5-105 Fracture of bar 2B near the base of the column



Figure 5-106 Experimental extent of plasticity

5.8.3. Force-displacement response

The hysteretic response of the column at all applied ductility levels and indications of bar fracture are shown in Figure 5-107. The specimen presented bar fractures in three bars during the test. The first bar fractured during the second pushing cycle of μ_5 , which corresponded to the extreme top bar (2T), resulting in a 20% drop in strength in the pushing direction (-). A second bar fractured during the third pulling cycle of μ_5 , corresponding to the bottom extreme bar 2B, and caused a 20% drop in strength in the pulling direction (+). Lastly, the top bar 3T fracture in the third pushing cycle of μ_5 . The test was finalized at this point, as the strength in the pushing direction had decreased by 60% from its maximum. These observations are also evident in the force-displacement envelopes of the columns shown in Figure 5-108.

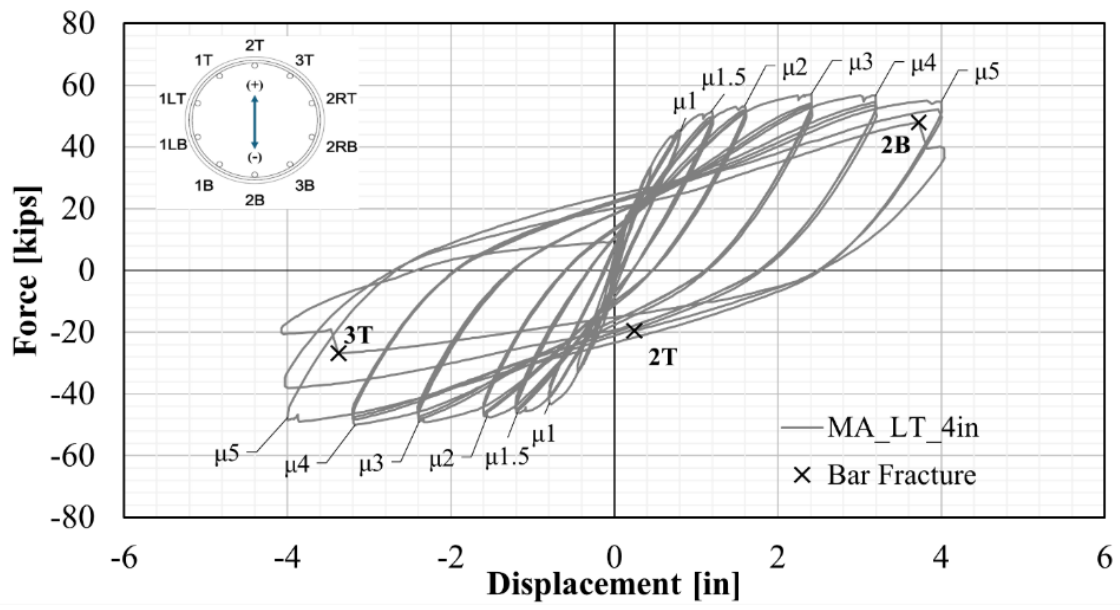


Figure 5-107 Hysteretic response of Column 5: MA_LT_4in

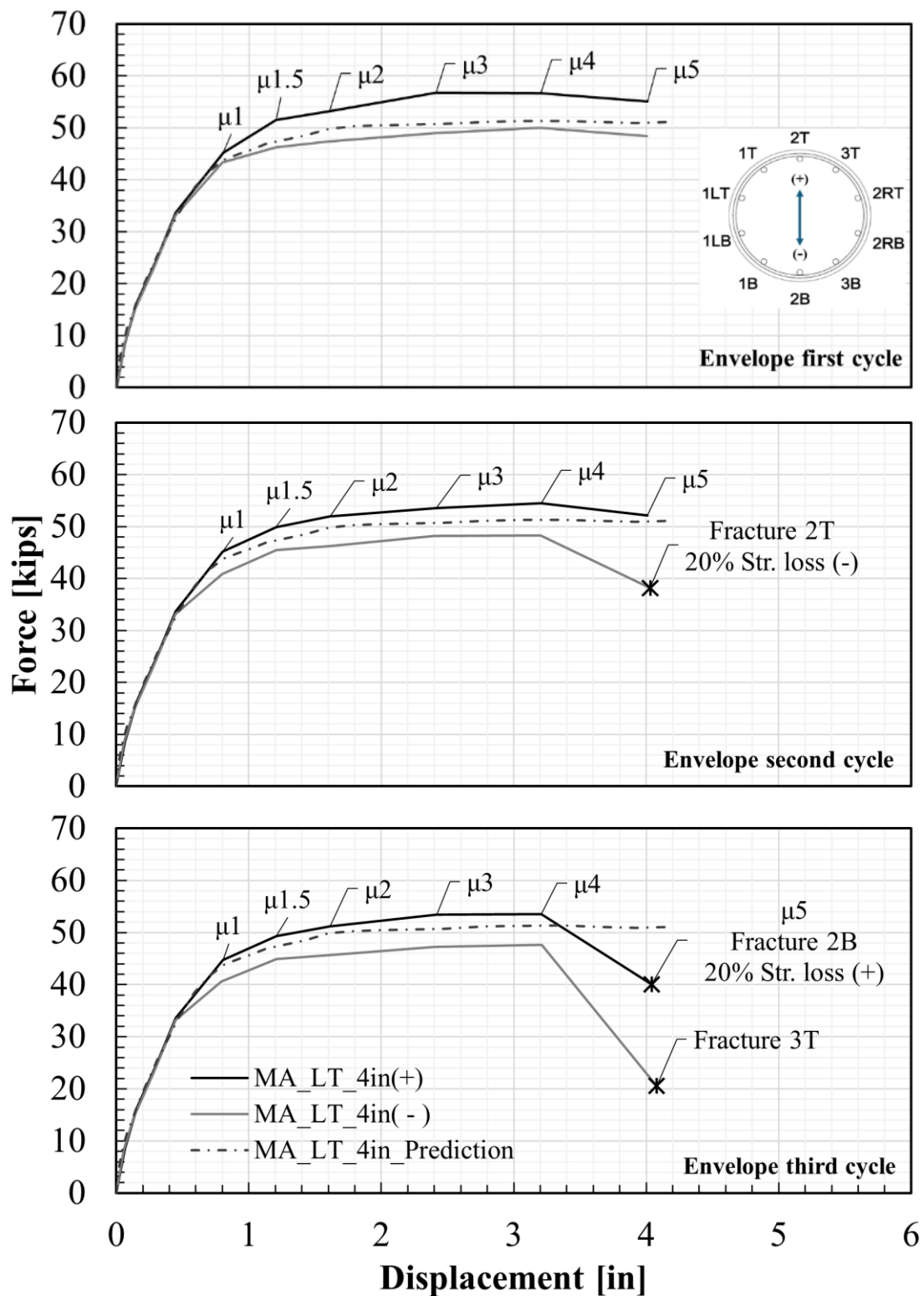


Figure 5-108 Envelope of the Force-displacement response of the RC column specimen #5

5.8.4. Curvature Profile

The low-level and high-level curvature profiles for Column5:MA_LT_4in are presented in Figure 5-109 and Figure 5-110, respectively. At low-level curvatures, the maximum curvature was concentrated in the third gage, corresponding to a column location of approximately 17.5 in above the footing. However, these values were mainly within the first yield curvature, just exceeding it at the third gage of the predicted first yield loading cycle in the pushing direction (dashed lines). The initial curvature distribution shifted as the column underwent higher displacement. The greater values of curvature were concentrated at the base of the column for the pulling directions and on the second gauge for the pushing direction, presenting an uneven distribution between the two loading directions in the high-level curvature profile (Figure 5-110). Higher curvature values were observed in the pulling direction (solid lines), which correlates well with most of the damage observed at the top of the column.

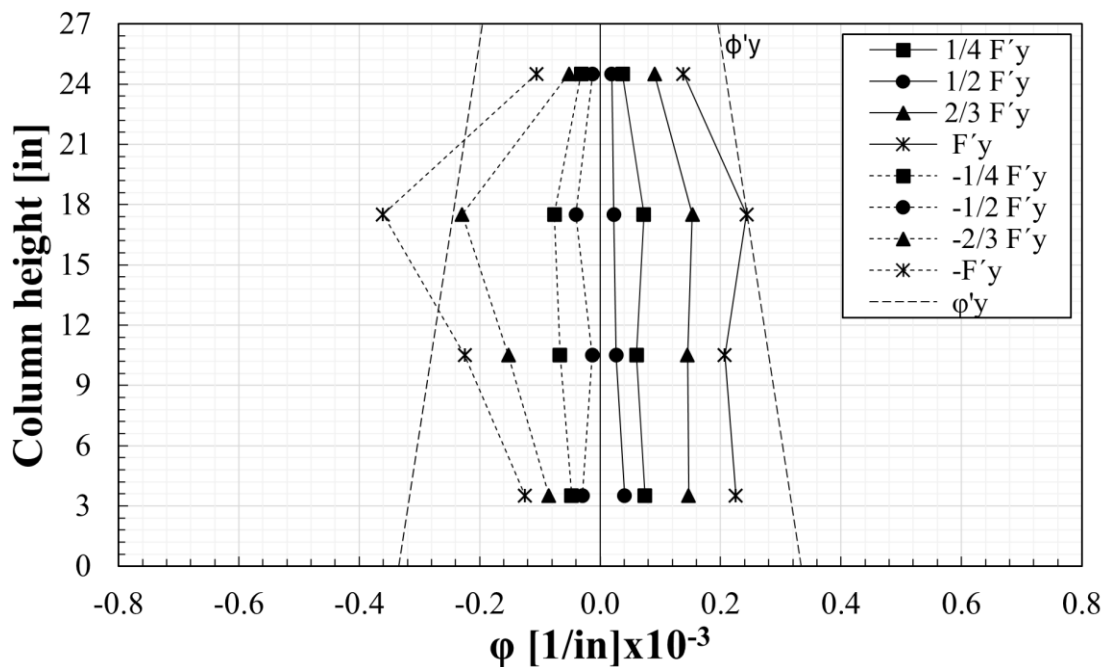


Figure 5-109 Low-level curvature profile of Column 5: MA_LT_4in

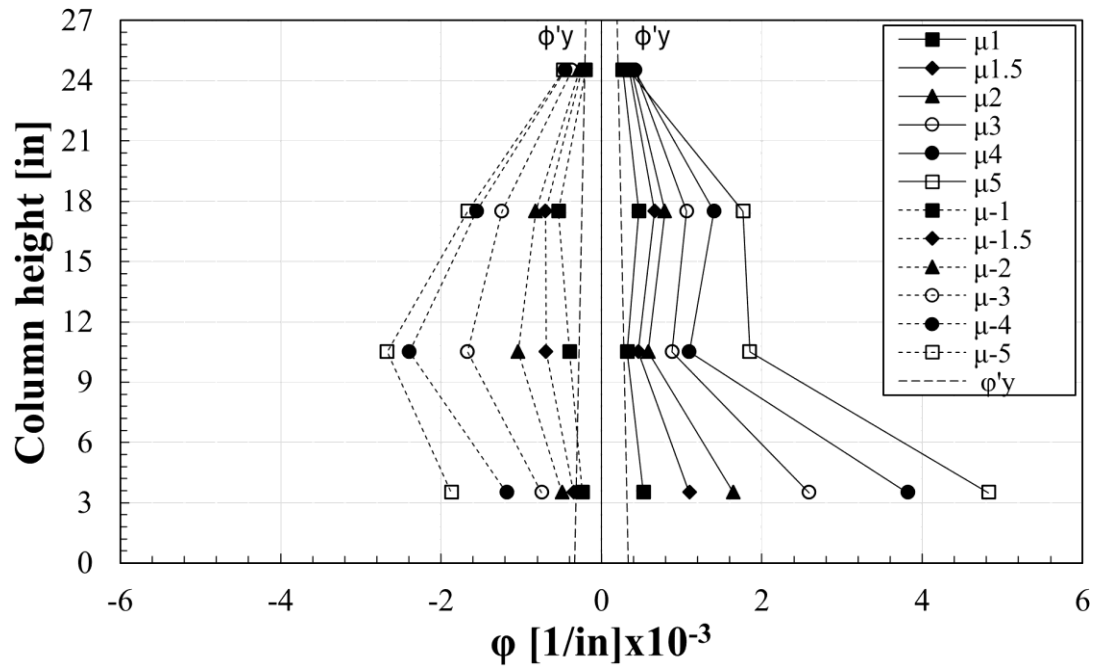


Figure 5-110 High-level curvature profile of Column 5: MA_LT_4in

5.9. Experimental response COL6: QST LT 4IN

5.9.1. Summary

The final test was performed at low temperature on an 18-inch-diameter column reinforced with 10 #6 QST ASTM A706 grade 80 bars and a 4-inch spiral separation. The column was conditioned for about 20 hours at a set temperature of -40°F, reaching -33°F at the reinforcing steel, and had an average temperature of -32°F during the test (Figure 5-111). This specimen was conditioned for less time than the other low-temperature specimens, which were conditioned for approximately 37 hours, due to operational time constraints in the environmental chamber. However, based on temperature records from previous columns, the specimen's temperature began to stabilize after about 20 hours of conditioning, which contributes to the consistency of the test condition.

During the test, the specimen reached a maximum force of 60 kips and exhibited a 20% strength loss during the first cycle of μ5 in both directions due to the fracture of the

extreme bars 2T and 2B. The column was loaded until the second push cycle of $\mu 5$, at which point a drop in strength of about 50% was observed. The summary of the column parameters is presented in Table 5-19.

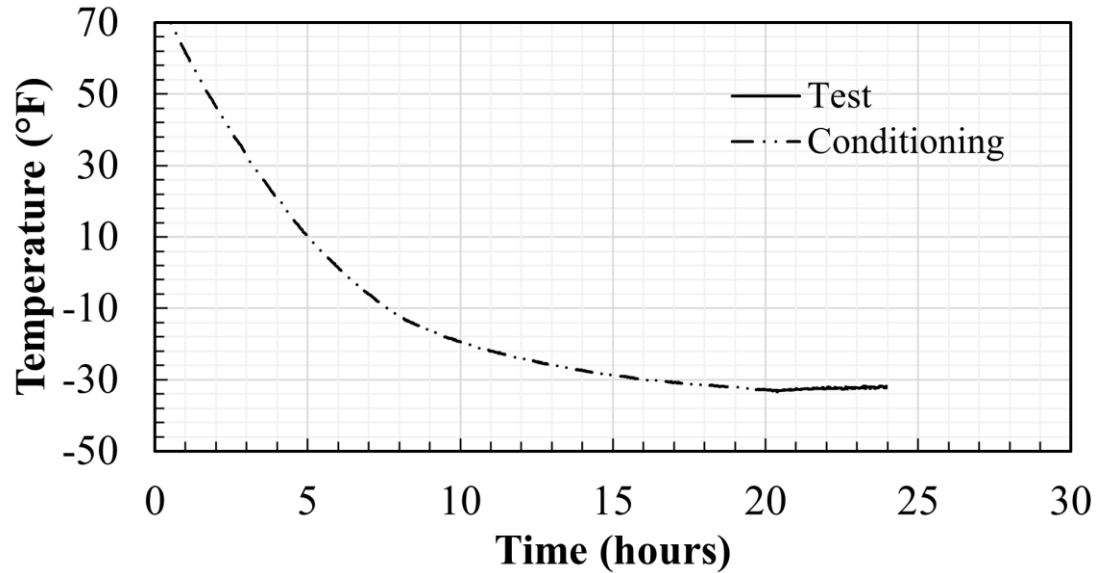


Figure 5-111 Temperature record of column 6:QST_LT_4in during the conditioning time and testing

Table 5-19 Summary of column parameters

Column parameters	
Longitudinal Steel:	10 #6 ASTM A706 Gr.80 ($\rho_l=1.74\%$) QST
Transverse Steel:	#3 ASTM A706 Gr.80 @ 2" ($\rho_t=1.23\%$)
Longitudinal Steel Nominal Yield Strength RT:	86.3 ksi
Longitudinal Steel Nominal Yield Strength LT:	92.3 ksi
Transverse Steel Nominal Yield Strength RT:	91.2 ksi
Transverse Steel Nominal Yield Strength LT:	97.6 ksi
Concrete Strength_RT:	4.9 ksi
Concrete Strength_LT:	7.4 ksi
Axial Load:	123 ksi
Axial Load Ratio:	9.9%
First Yield Displacement:	0.55 in
F _y	39.2 kip
Nominal Moment Capacity:	211 kip-ft
Equivalent Yield Displacement:	0.7 in
Maximum Lateral Force:	60 kips
Displacement at 20% Strength loss:	4 in
Failure Mode:	Fracture of buckled bar

5.9.2. Test observations

The first hairline cracks along the plastic hinge appeared at $3/4 F_y$ and continued expanding throughout the test. Concrete crushing was observed in the first pulling cycle of μ_4 . At the third pulling cycle of μ_4 , buckling of the top extreme bar 2T was observed. The first bar fracture in the first pulling cycle of μ_5 , which corresponded to the extreme bottom bar 2B. This was followed by the fracture of the top extreme bar 2B in the first pushing cycle of μ_5 . The fracture of the extreme bars resulted in 20% strength degradation in both loading directions of the column.

A third bar (1B) fractured at the bottom of the column in the second pulling cycle of μ_5 , corresponding to the bar adjacent to the extreme bar 2 B. This fracture resulted in a total strength reduction of about 50% of the column's maximum strength, and the test was stopped after the completion of the second pushing cycle of μ_5 . Figure 5-112 to Figure 5-123 show the damage progression of the column. Table 5-20 presents the summary of the damage observations from the test of the sixth column specimen.

Table 5-20 Column 6 damage progression summary

Damage observations	
Initiation of cracking:	$3/4 f_y$
Core Concrete Crushing:	μ_4^{+1}
Longitudinal Bar Buckling Top:	μ_4^{+3}
Longitudinal Bar Buckling Bottom :	None visible
Longitudinal Bar Fracture Top:	μ_5^{-1}
Longitudinal Bar Fracture Bottom:	μ_5^{+1}
20% Strength loss (+):	μ_5^{-1}
20% Strength loss (-):	μ_5^{+1}
(+) Pulling, (-) Pushing	

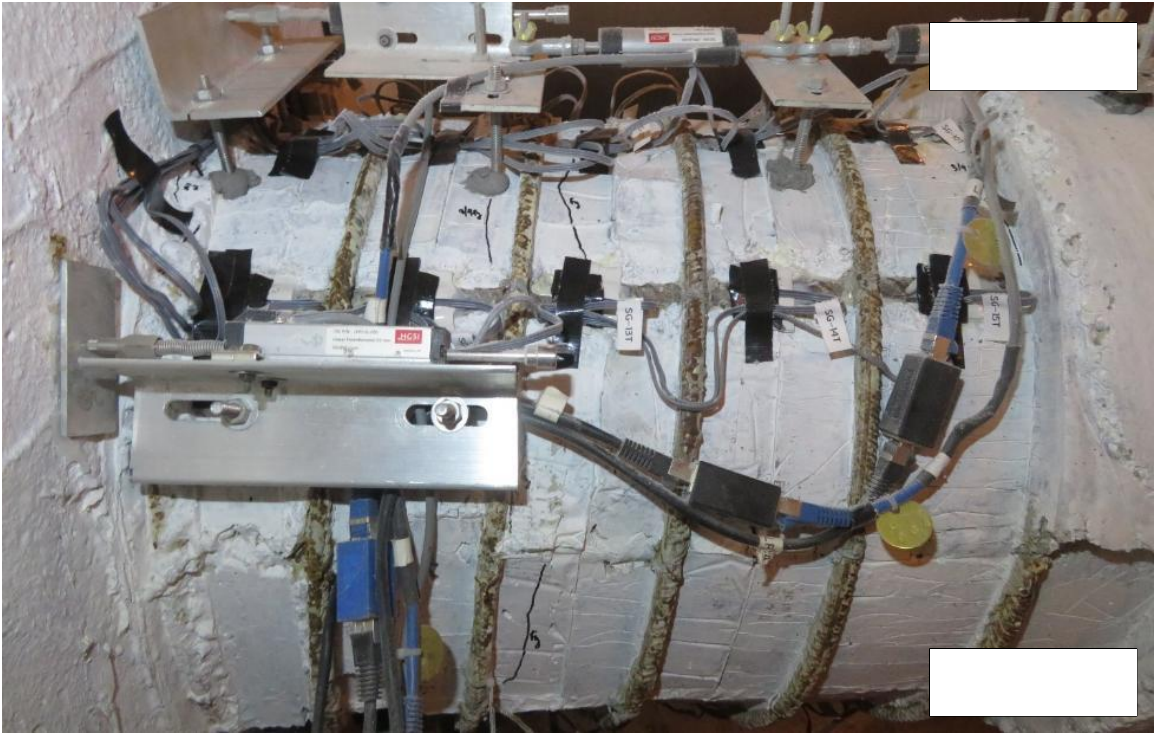


Figure 5-112 F_y (pushing). Hairline cracks formed along the plastic hinge in the first cycle of $3/4 F_y$ and continued expanding.



Figure 5-113 Third pushing cycle of μ_1 : Extension of cracks



Figure 5-114 Third pushing cycle of $\mu 1.5$: Extension of cracks and formation of some shear cracks



Figure 5-115 Third pulling cycle of $\mu 3$: Concrete flaking near the base of the column.



Figure 5-116 First pulling cycle of $\mu 4$: Concrete crushing near the base of the column



Figure 5-117 Third pulling cycle of $\mu 4$: Buckling of top extreme bar 2T



Figure 5-118 First pulling cycle of $\mu 5$: Bar fracture of bottom extreme bar 2B



Figure 5-119 First pushing cycle of $\mu 5$: Bar fracture of top extreme bar 2T



Figure 5-120 Second pulling cycle of $\mu 5$: Bar fracture of adjacent bar to top extreme bar 1B



Figure 5-121 Damage at the top of the column along the plastic hinge: Fracture of bar 2T and concrete core crushing

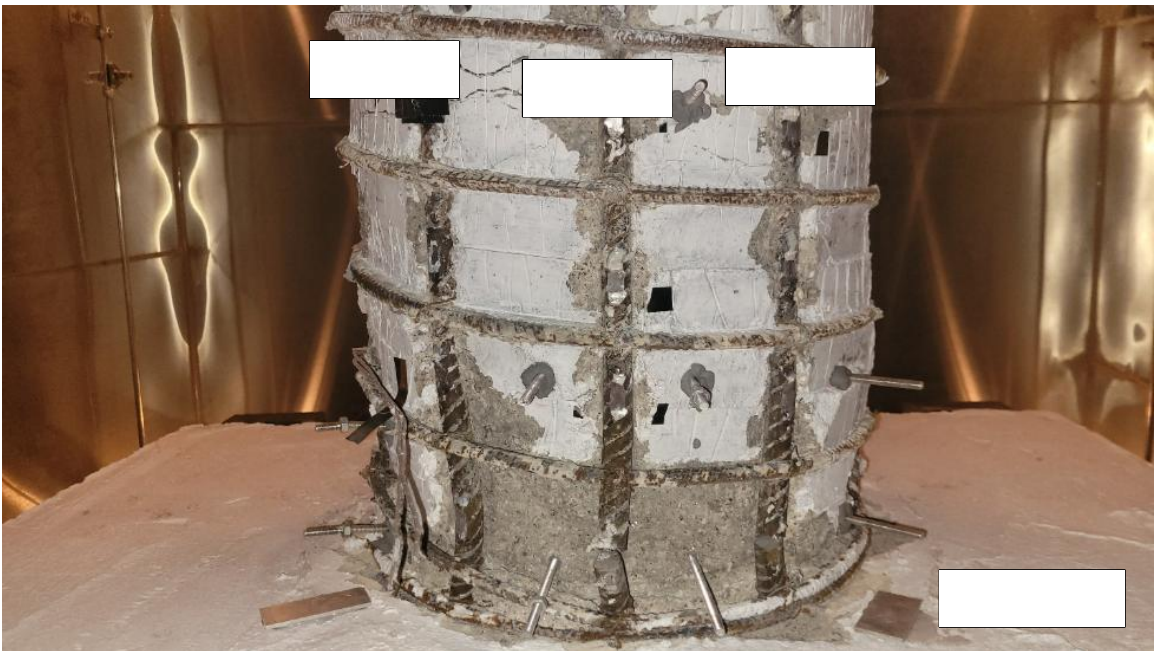


Figure 5-122 Damage at the bottom of the column along the plastic hinge: Fracture of bars 2B and 1B, and concrete core crushing



Figure 5-123 Experimental extent of plasticity.

5.9.3. Force-displacement response

Figure 5-124 presents the hysteretic response of the column at all applied ductility levels and indications of bar fracture. The specimen presented fractures in three bars. The bottom extreme bar 2B fractured in the first pulling cycle of $\mu 5$, followed by the fracture of the top extreme bar 2T, which caused a 20% drop in strength in the pushing and pulling direction. Lastly, the bottom extreme bar (1B) fractured in the second pulling cycle of $\mu 5$, resulting in significant strength degradation. The test was finalized after the completion of the second pushing cycle of $\mu 5$. These observations are also evident in the force-displacement envelopes of the columns shown in Figure 5-125.

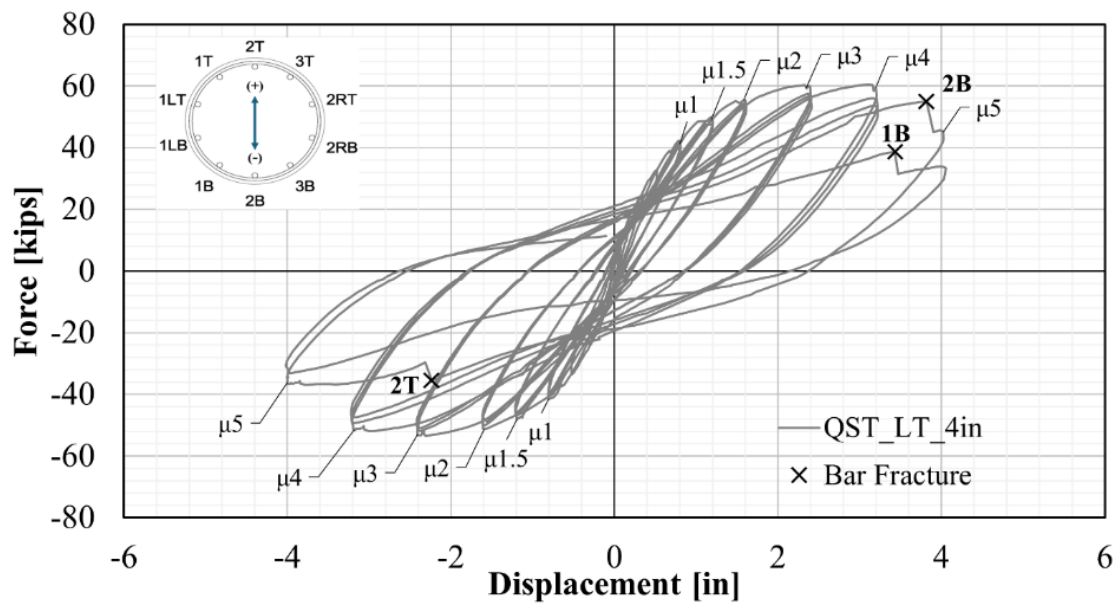


Figure 5-124 Hysteretic response of Column 6: QST_LT_4in

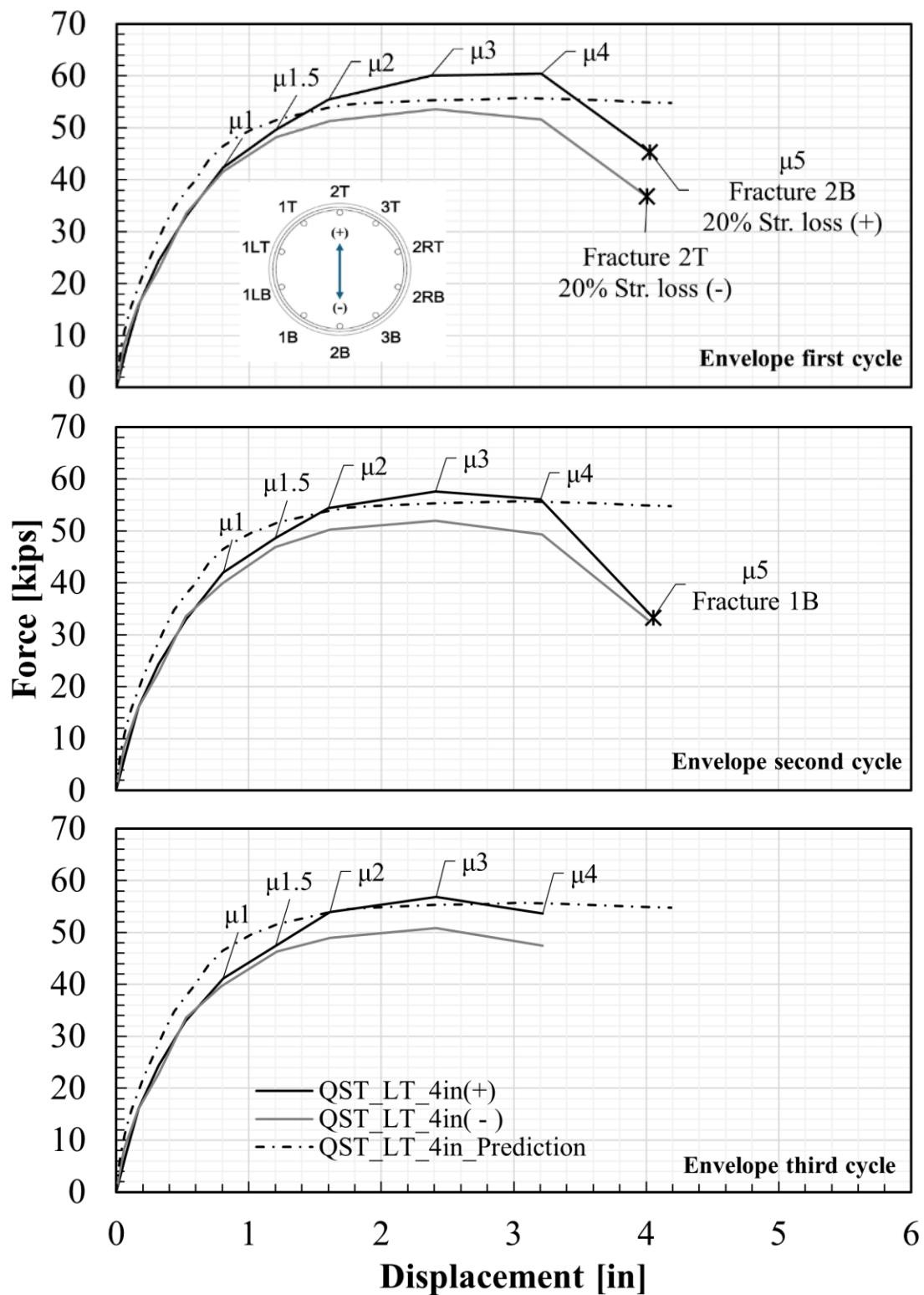


Figure 5-125 Envelope of the Force-displacement response of the RC column specimen #6

5.9.4. Curvature Profile

Figure 5-126 and Figure 5-127 present the low- and high-level curvature profiles for Column6:QST_LT_4in. Similar to the previous column, the maximum curvature was concentrated in the third gage, corresponding to a location 17.5 in above the footing, at low-level curvature, with values mainly within the first yield curvature, just exceeding it at the third gage of the first yield loading cycle in both loading directions. At the high-level curvature profile (Figure 5-110), the initial concentration of curvature was maintained until μ_2 , after which the distribution shape shifted to greater values of curvature concentrated at the base of the column. Higher curvature values were observed in the pushing direction (dashed lines), which correlates well with most of the damage observed at the bottom of the column. The curvature distribution corresponding to μ_5 was not presented for either loading direction because the column damage affected the instrument support, resulting in unreliable results for this ductility level.

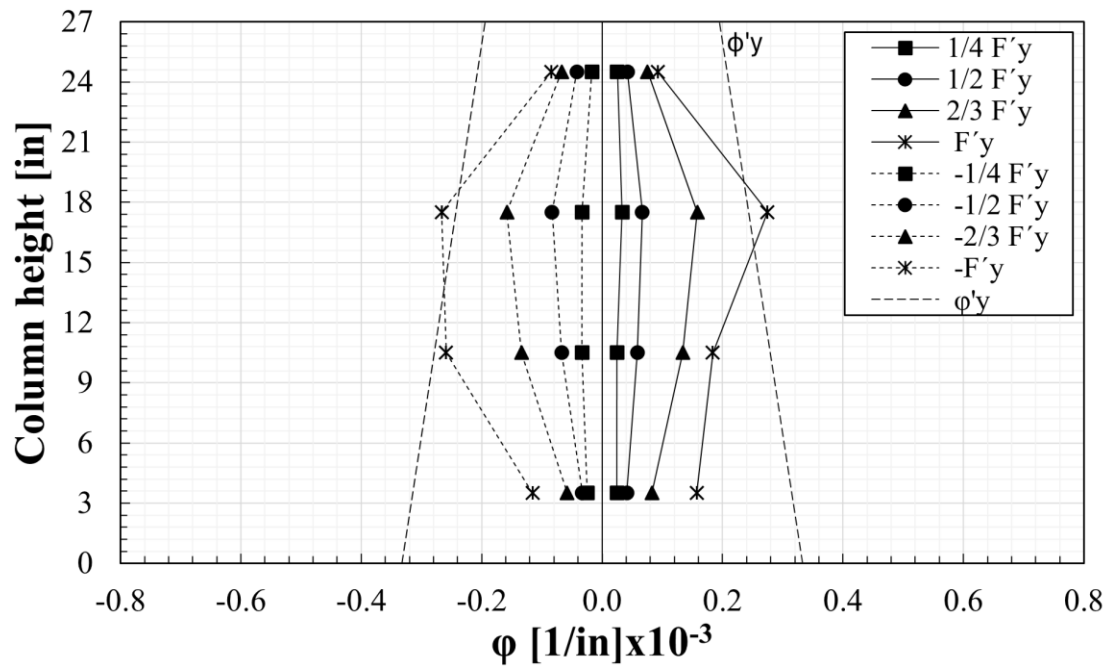


Figure 5-126. Low-level curvature profile of Column 6: QST_LT_4in.

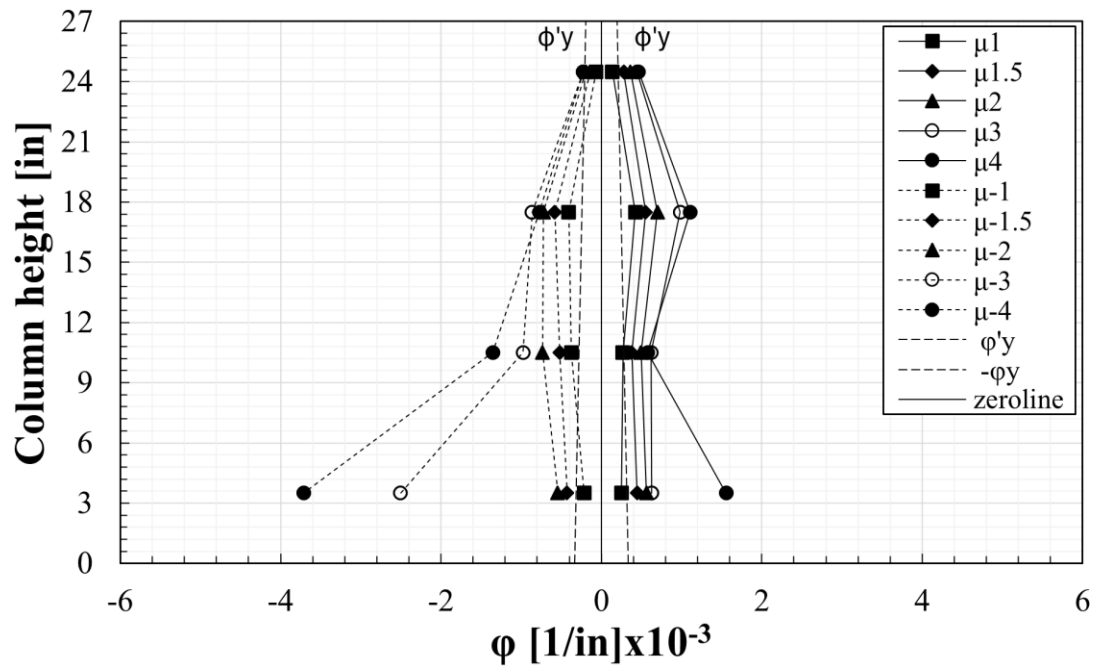


Figure 5-127. High-level curvature profile of Column 6: QST_LT_4in.

5.10. IMPACT OF LOW TEMPERATURE ON HSRS

5.10.1. Low temperature impact on the seismic behavior of RC columns reinforced with grade 80 MA steel

Tests 1 and 3 corresponded to column tests performed on RC Columns reinforced using MA steel. The first was conducted at room temperature, while the last was tested at low temperature. The hysteretic response of the two specimens is shown in Figure 5-128, and the first and third cycles' envelope of the force-displacement response is presented in Figure 5-129 and Figure 5-130. Overall, both columns reached a displacement of 4 in before they achieved a 20% strength loss due to bar fracture, and both columns were loaded until a displacement of 4.8 in, achieving μ_6 . The low-temperature specimen exhibited strength approximately 17% higher than the room-temperature specimen, with maximum strengths of 59 kips and 50 kips, respectively.

The main differences observed between the two specimens were the damage progression and the post-buckling behavior. During the test, the room-temperature specimen showed cracking earlier than the low-temperature specimen. In addition, more cracks and damage to the concrete core were observed in the room-temperature specimen, as shown in Figure 5-133. These differences in the concrete damage could be explained by the increased strength at low temperatures, where the concrete achieved 42% higher strength than at room temperature. The second main difference was in post-buckling behavior, where the low-temperature specimen exhibited a significant reduction in bending strain capacity. For the low-temperature specimen, visible bar buckling was observed in the first pulling cycle of ductility 5, followed by immediate bar fracture in the subsequent pushing cycle. In contrast, the room-temperature specimen exhibited visible bar buckling in the third pulling cycle of μ_4 and bar fracture in the third pushing cycle of ductility 5, allowing 3 complete cycles of

loading between buckling and fracture. The comparison of the visible buckling and fracture of the top extreme bar for both specimens is shown in Figure 5-130 and Figure 5-131. These observations correlate well with the material-level experiments, in which buckled bar tension test results on MA bars showed a lower bending strain capacity at low temperature than at room temperature. The reduction in the bending strain capacity at low temperatures results in fractures that are more brittle in nature at low temperatures compared to bars subjected to the same amount of buckling at room temperature.

Despite the reduction in the bending strain capacity of the longitudinal bars in the low-temperature specimen, the column exhibited a global response comparable to that of the room-temperature specimen. The similarity in global behavior is explained by the increase in the strength of concrete and transverse steel, which provided greater confinement of the longitudinal bars under low-temperature conditions, thereby delaying the onset of buckling.

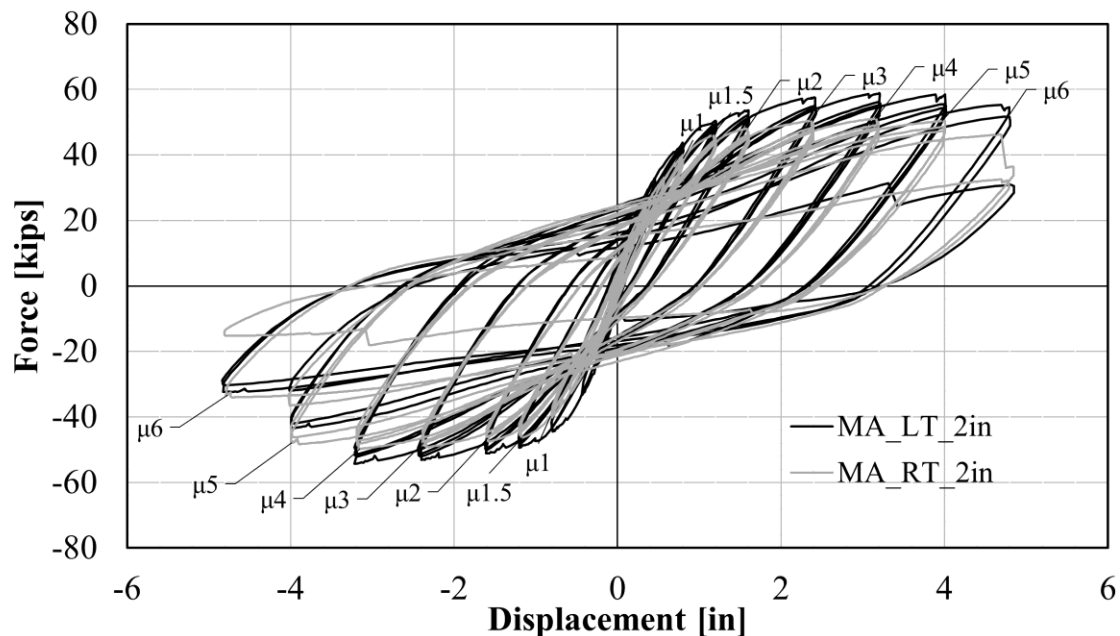


Figure 5-128 Comparison of the hysteretic response of Columns 1 and 3: MA_RT_2in vs. MA_LT_2in

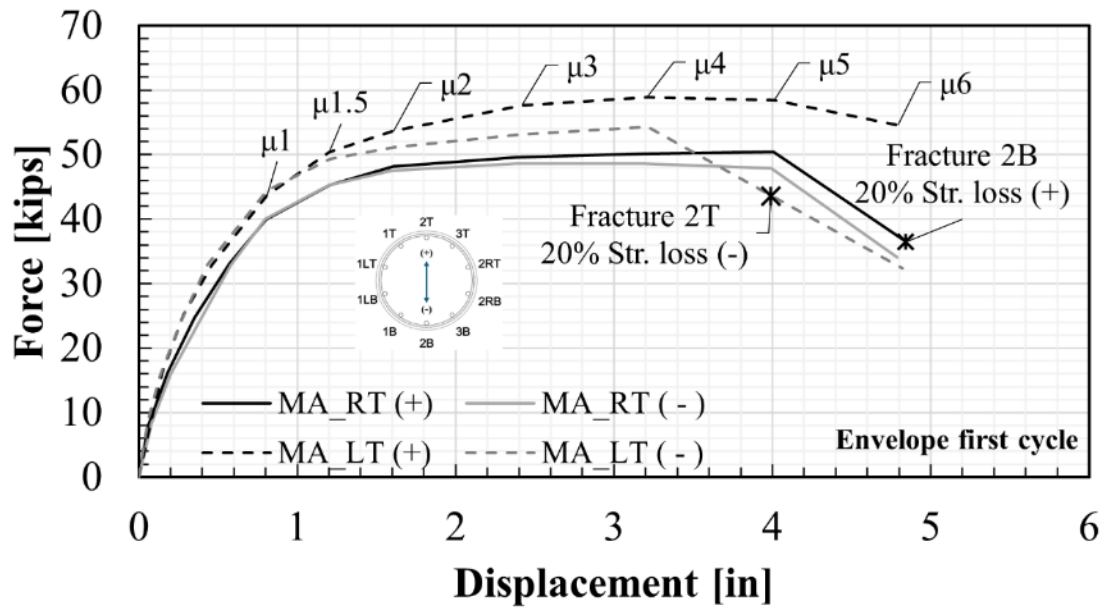


Figure 5-129 Comparison of the first cycle envelope of the force displacement response of Columns 1 and 3:
MA_RT_2in vs. MA_LT_2in

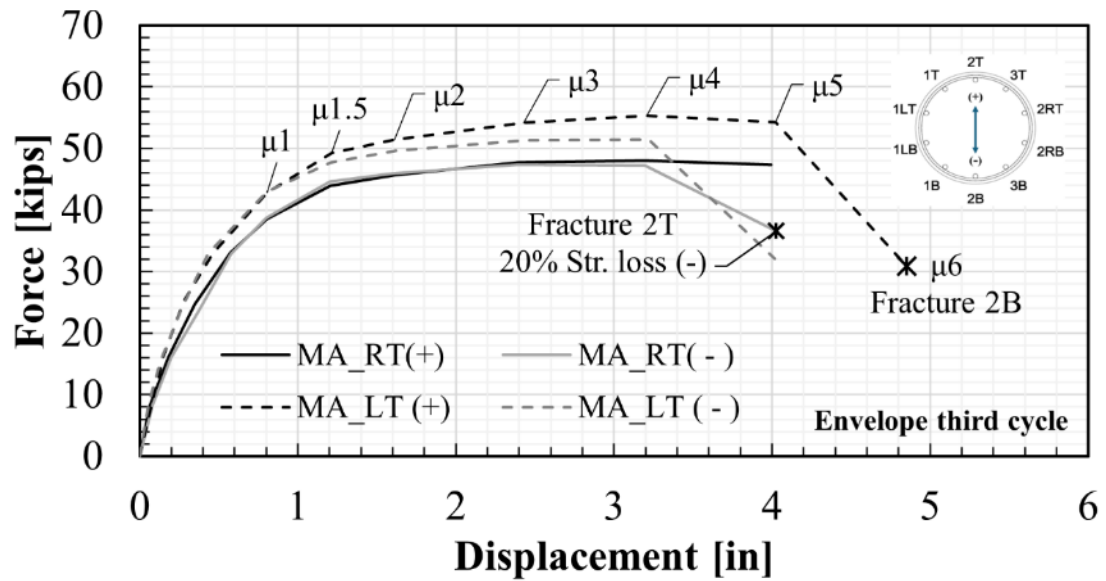


Figure 5-130 Comparison of the third cycle envelope of the force displacement response of Columns 1 and 3:
MA_RT_2in vs. MA_LT_2in

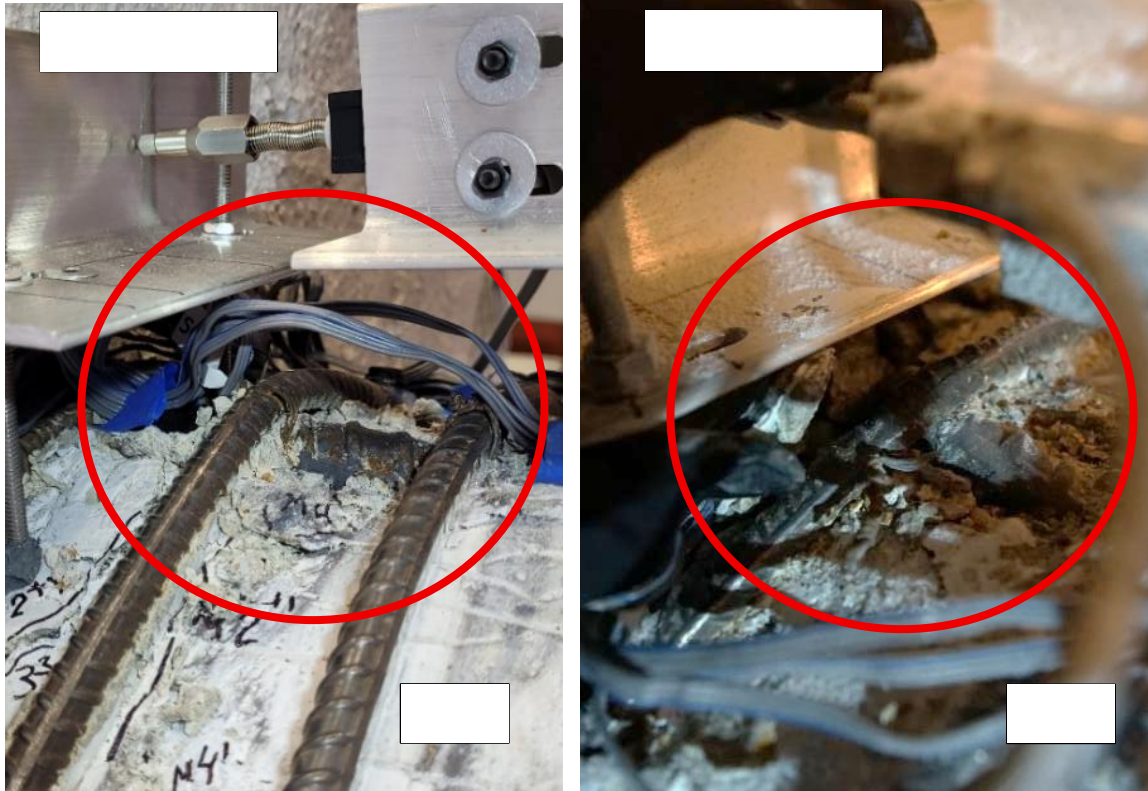


Figure 5-131 Visible buckling in the top extreme bar for the MA columns tested at room and low temperatures

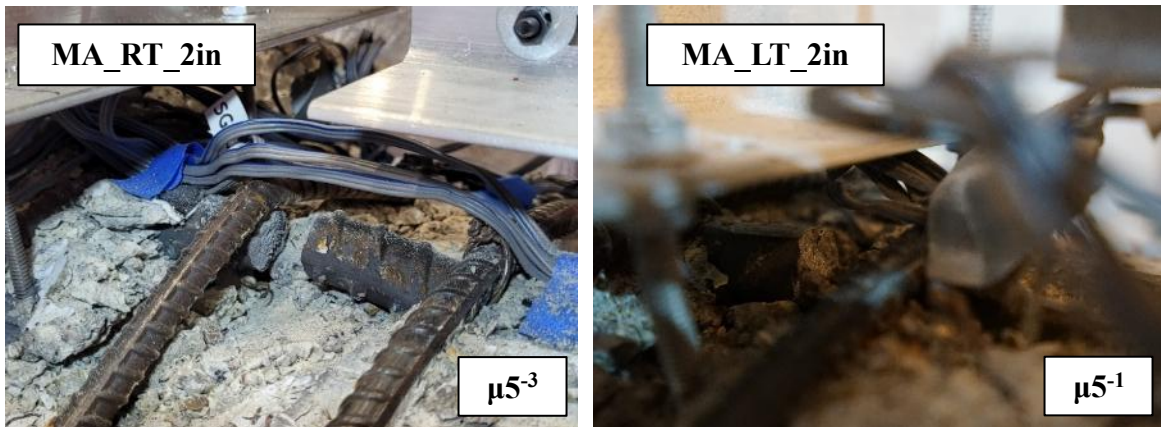


Figure 5-132 Top extreme bar fractures for the MA columns tested at room and low temperatures

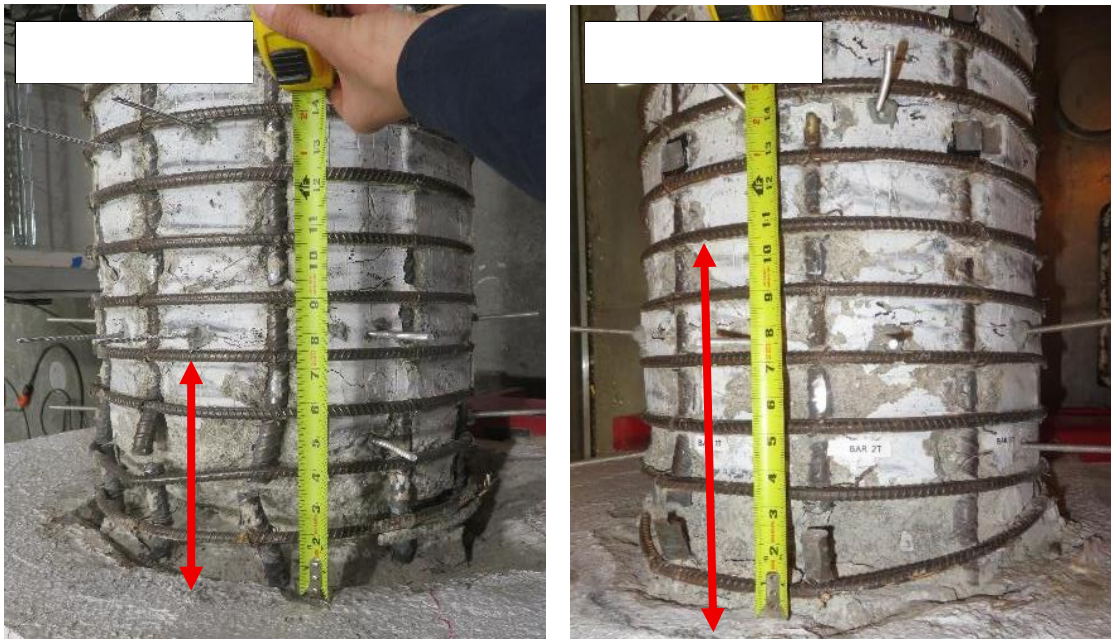


Figure 5-133 Concentration of concrete crushing at the base of the columns

Figure 5-134 and Figure 5-135 present the comparison between the curvature distribution of Columns 1 and 3 at μ_1 and μ_4 , respectively. At μ_1 , the curvature profile in the pushing direction (dashed line) remained similar between the low- and room-temperature specimens. In the pulling direction (solid line), the room-temperature column exhibited higher curvature near the base than the low-temperature specimen (Figure 5-135). At μ_4 , both specimens presented a symmetrical curvature distribution along the column height, with the maximum curvature values obtained at the base of the column. The room-temperature specimen showed greater curvature at the base of the column than the low-temperature column. In addition, the low-temperature column exhibited a slightly larger spread of plasticity than the room-temperature column, as it showed larger curvature values at the second gage than its room-temperature pair. These observations are consistent with the final statement of the columns displayed in Figure 5-133 where crushing extended over a larger distance in the low-temperature specimen, whereas it was more severe and concentrated in the room-temperature column.

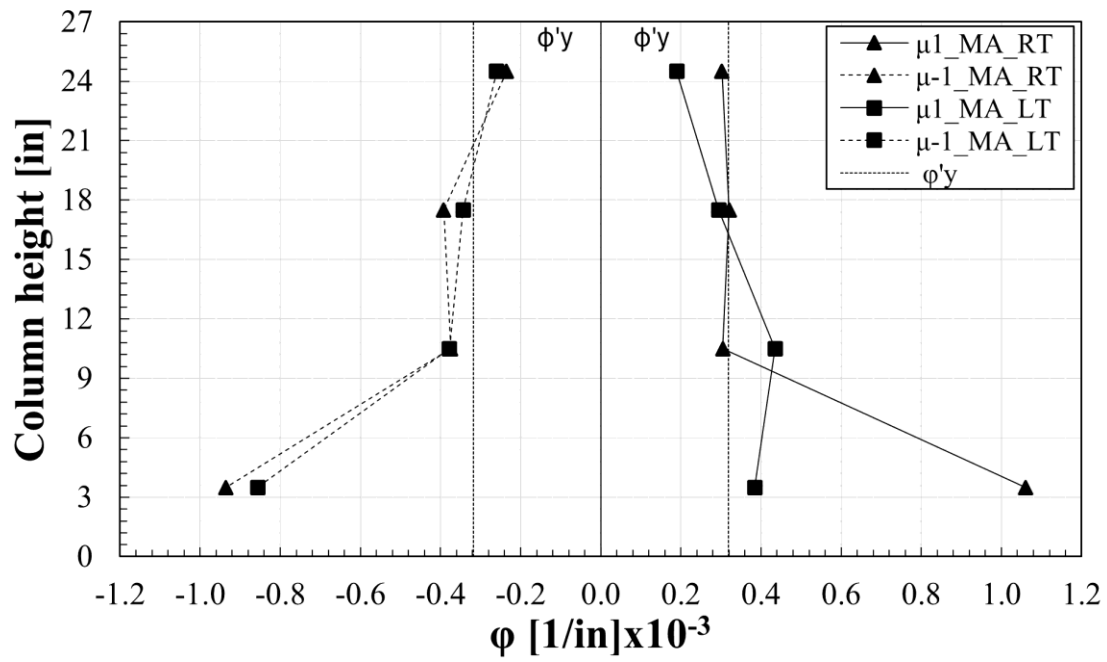


Figure 5-134 Comparison of the curvature distribution at μ_1 of Columns 1 and 3: MA_RT_2in vs. MA_LT_2in

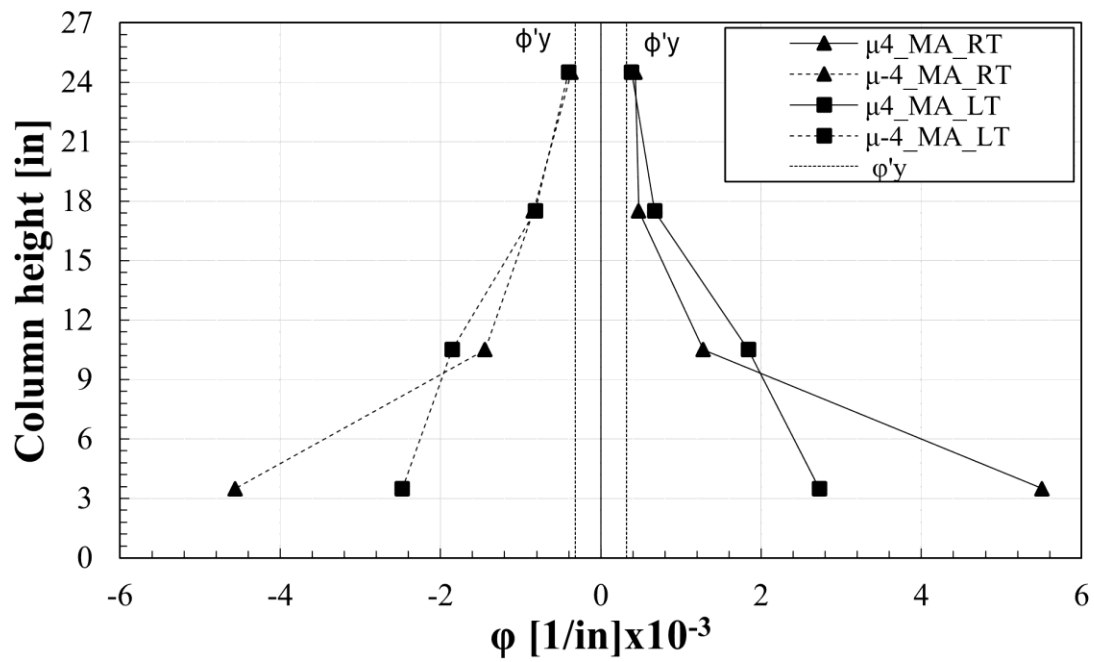


Figure 5-135 Comparison of the curvature distribution at μ_4 of Columns 1 and 3: MA_RT_2in vs. MA_LT_2in

5.10.2. Low temperature impact on the seismic behavior of RC columns reinforced with grade 80 QST steel

Tests 2 and 4 corresponded to column tests performed on RC Columns reinforced using QST steel, conducted at room and low temperatures, respectively. Figure 5-136 shows the comparison of the hysteretic responses of the two specimens. The force-displacement envelopes of the first and third cycles of the columns are shown in Figure 5-137 and Figure 5-138. It was observed that both specimens exhibited 20% strength degradation in $\mu 5$; however, the specimen tested at low temperature showed greater strength degradation at this ductility level than the room-temperature column, as more bars fractured at this point. By contrast, the column tested at room temperature achieved greater residual strength in the third cycle of $\mu 5$, and it was loaded for an additional cycle in $\mu 6$. These results indicate that temperature affected the column's displacement capacity, with slightly lower displacement capacity at low temperatures.

Related to bar buckling and bar fracture, both specimens presented an indication of the extreme top bar buckling at the same ductility level (Figure 5-139). However, the fracture in the low-temperature specimen occurred during the first pushing cycle of $\mu 5$ and was more brittle. Figure 5-140 shows the fracture of the top extreme bars for the room and low temperature specimens, where it is also appreciated that the failure surface differs between the two specimens. The room-temperature specimen exhibited a diagonal fracture, typical of QST steel, whereas the low-temperature specimen exhibited a flat fracture. This change in the fracture surface was also observed in the material-level evaluation of QST bars, where bars subjected to higher buckling levels during the BBT test exhibited flatter failure surfaces as the failure mode shifted to brittle (Figure 5-141).

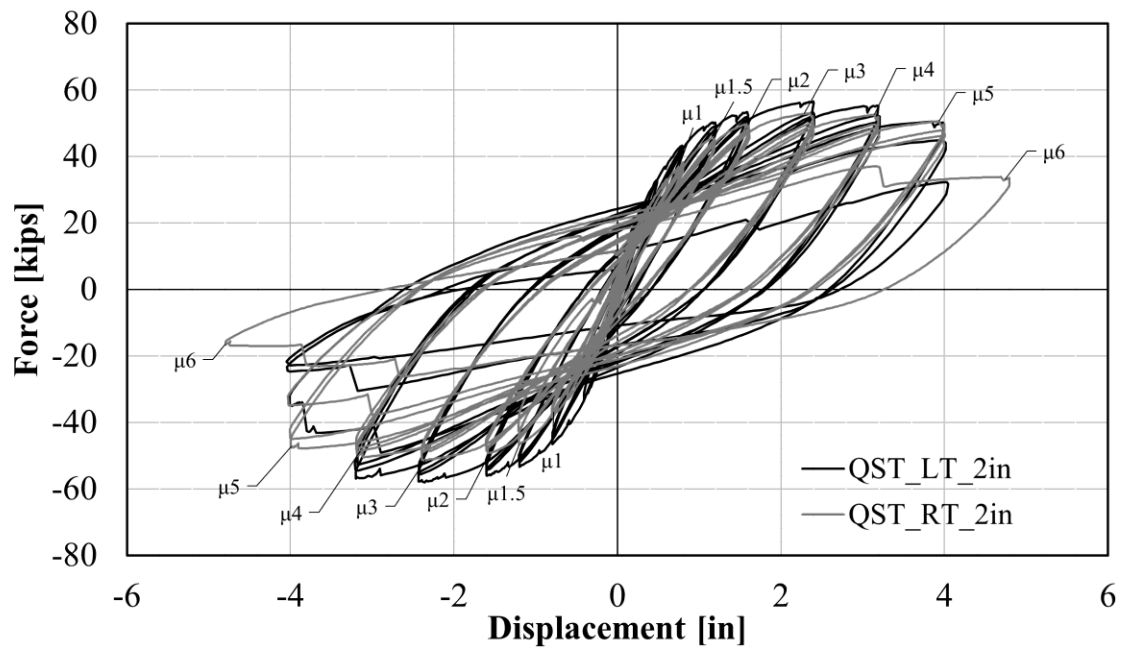


Figure 5-136 Comparison of the hysteretic response of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in

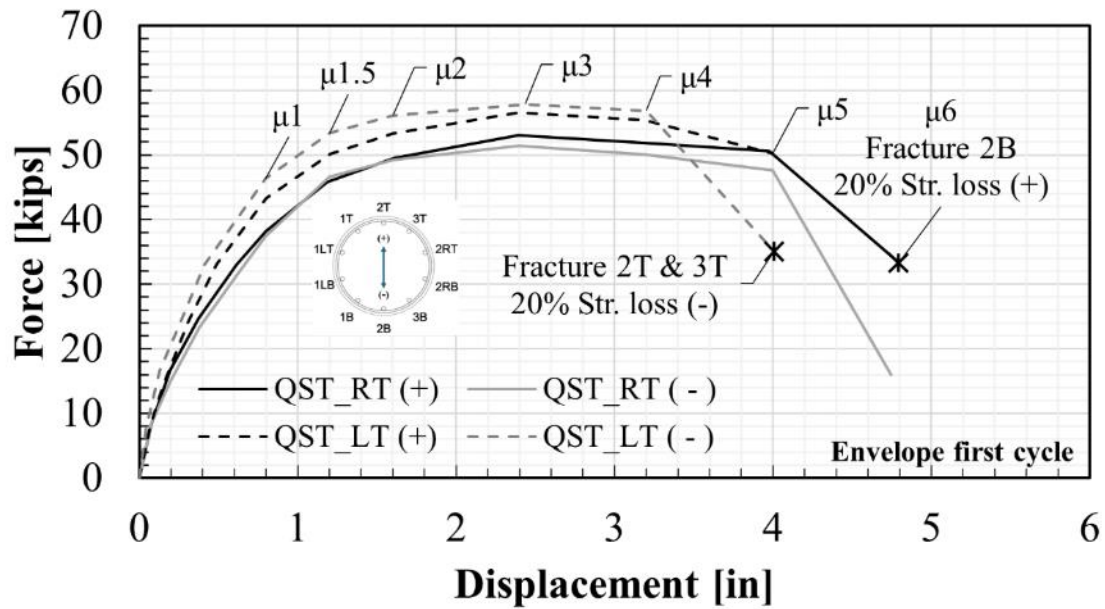


Figure 5-137 Comparison of the first cycle envelope of the force displacement response of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in

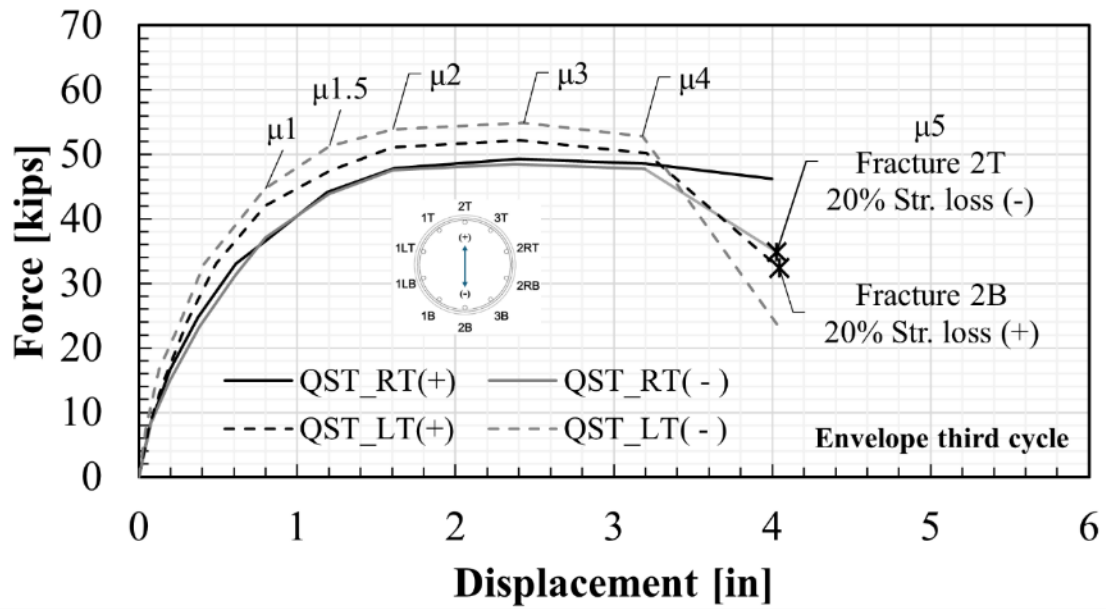


Figure 5-138 Comparison of the third cycle envelope of the force displacement response of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in.



Figure 5-139 Buckling of the top extreme bar in the QST columns, tested at room and low temperatures.



Figure 5-140 Fracture of the top extreme bar in the QST columns, tested at room and low temperatures.

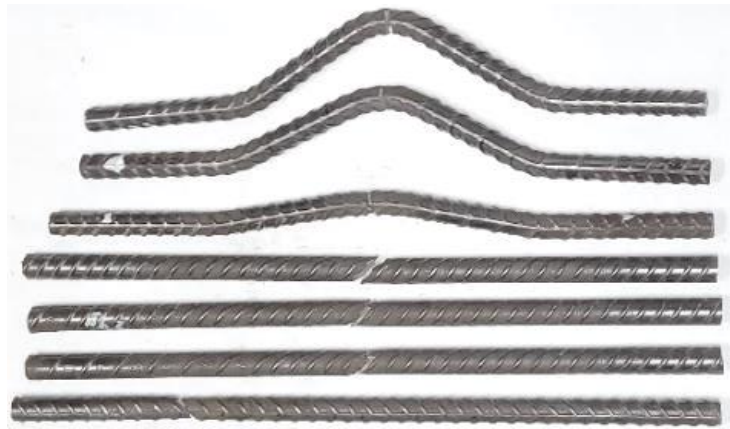


Figure 5-141 QST broken bars subjected to the BBT test at low temperature. Specimens were arranged from ductile to brittle failure modes (increasing levels of buckling at the top).

The comparison between the curvature distributions for Columns 2 and 4 at ductilities 1 and 4 is presented in Figure 5-142 and Figure 5-143, respectively. At both ductilities, the curvature distribution obtained for the QST specimens was similar at low temperature and room temperature. Both columns presented a symmetrical curvature distribution. At lower ductility, larger curvature values were concentrated in the first and second gages. In contrast, at higher ductility levels, both specimens exhibited larger curvature concentrations near the base of the columns. The differences in curvature values were minor, with slightly higher values at room temperature. No significant differences were appreciated in the spread of plasticity between the two specimens (Figure 5-144).

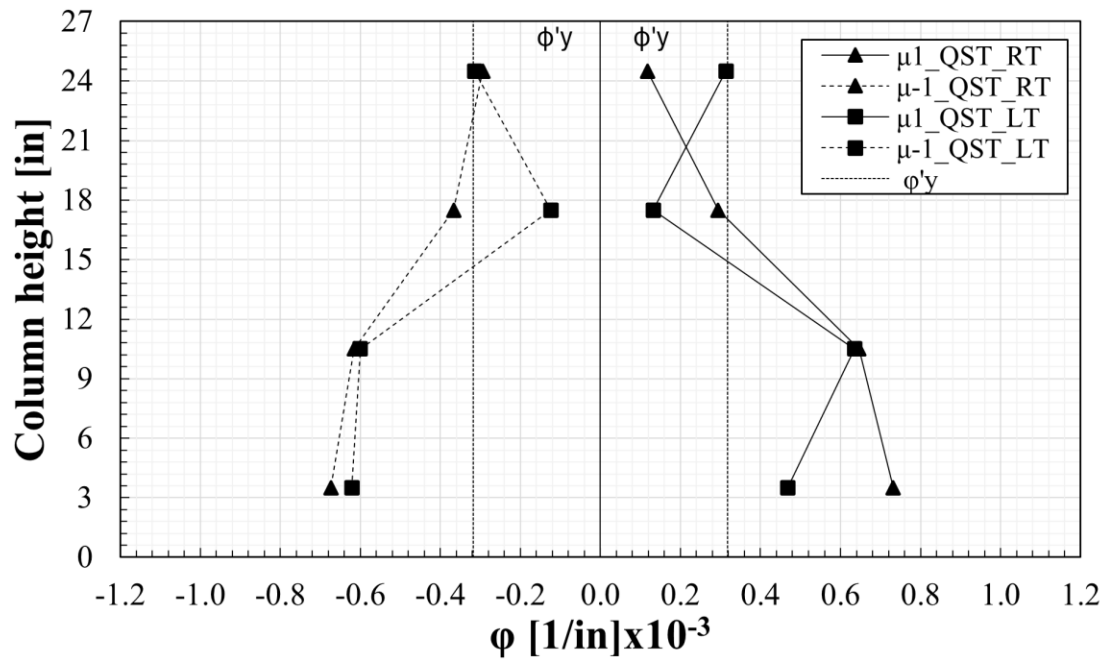


Figure 5-142 Comparison of the curvature distribution at μ_1 of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in

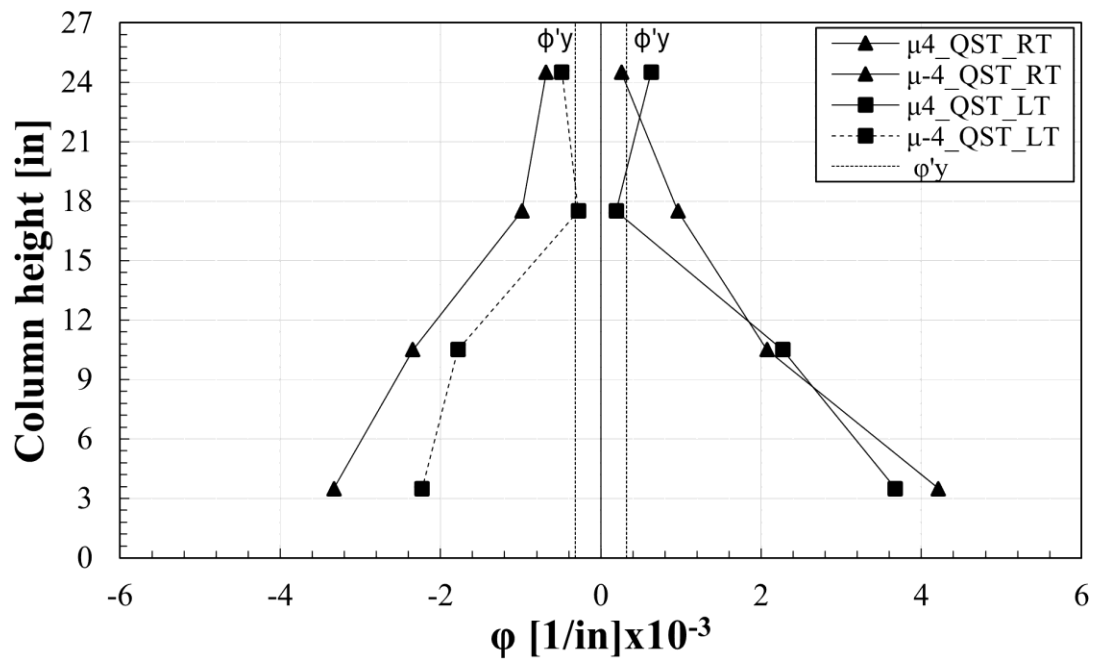


Figure 5-143 Comparison of the curvature distribution at μ_4 of Columns 2 and 4: QST_RT_2in vs. QST_LT_2in



Figure 5-144 Concentration of concrete crushing at the base of the columns

5.10.3. Manufacturing process impact on the seismic behavior of HSRS

5.10.3.1. Room temperature observations

Tests 1 and 2 corresponded to the room temperature control column tests. Both the MA and QST columns exhibited comparable behavior at room temperature. The columns were loaded to the first and second cycles of μ_6 , with the MA column achieving one additional complete loading cycle than the QST-reinforced column. The QST column exhibited slightly higher strength than the MA column, reaching a maximum force of 53 kips, compared to 50.41 kips obtained in the MA column. However, such minor differences are not unusual. Figure 5-145 presents a comparison of the hysteretic response of the MA and QST columns tested at room temperature. Both columns exhibited similar failure modes, with bar fracture of the top extreme bar occurring at the third pushing cycle for μ_5 and the first pulling cycle for μ_6 , resulting in 20% strength degradation in both loading directions

(pushing and pulling). A comparison of the force-displacement envelopes is presented in Figure 5-146 and Figure 5-147 for the first and third cycles of each ductility, respectively.

Bar buckling was more pronounced in the MA steel than in QST. Visible buckling was observed in the third pulling cycle of μ_4 for the MA column, while it was not visible for the QST column. However, the QST column exhibited significant concrete crushing along the longitudinal extreme bar at the first pulling cycle of μ_5 , which could be an indication of the initiation of buckling (Figure 5-148). The observations indicate that the MA and QST columns withstand approximately three complete cycles between buckling and bar fracture. The columns were loaded to ductility six, at which point significant strength degradation was observed, with 4 bars fracturing in each column. It was also observed that the MA bars exhibited a flat failure surface, whereas the QST bars exhibited a diagonal failure surface (Figure 5-149). This difference in the failure surface has also been observed in the material-level tests.

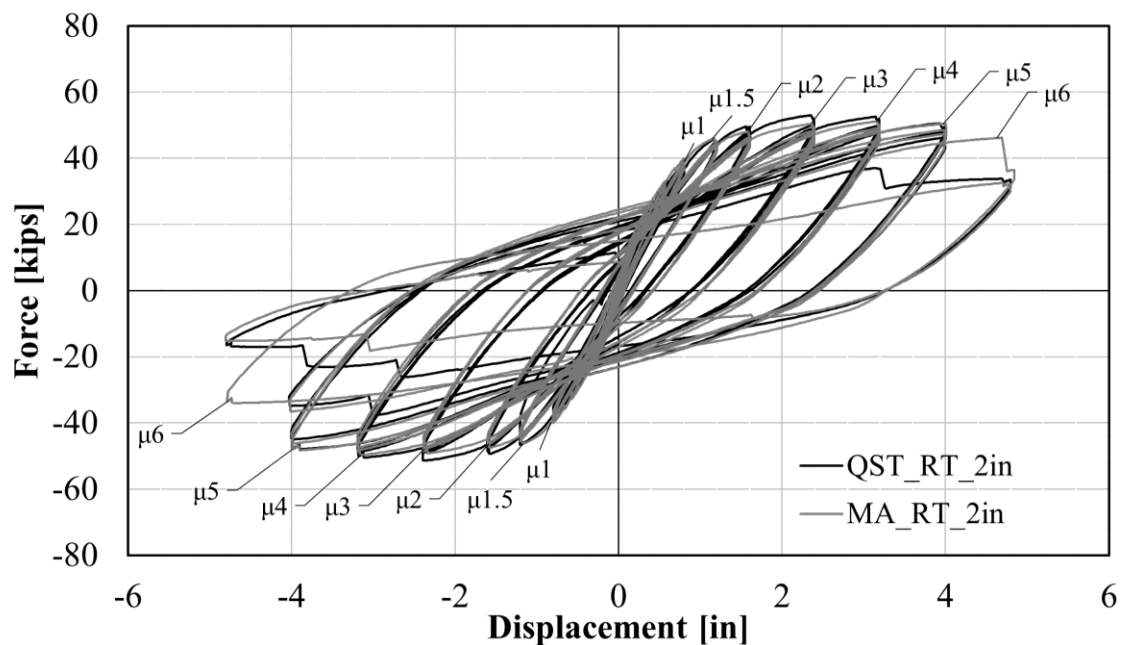


Figure 5-145 Comparison of the hysteretic response of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in.

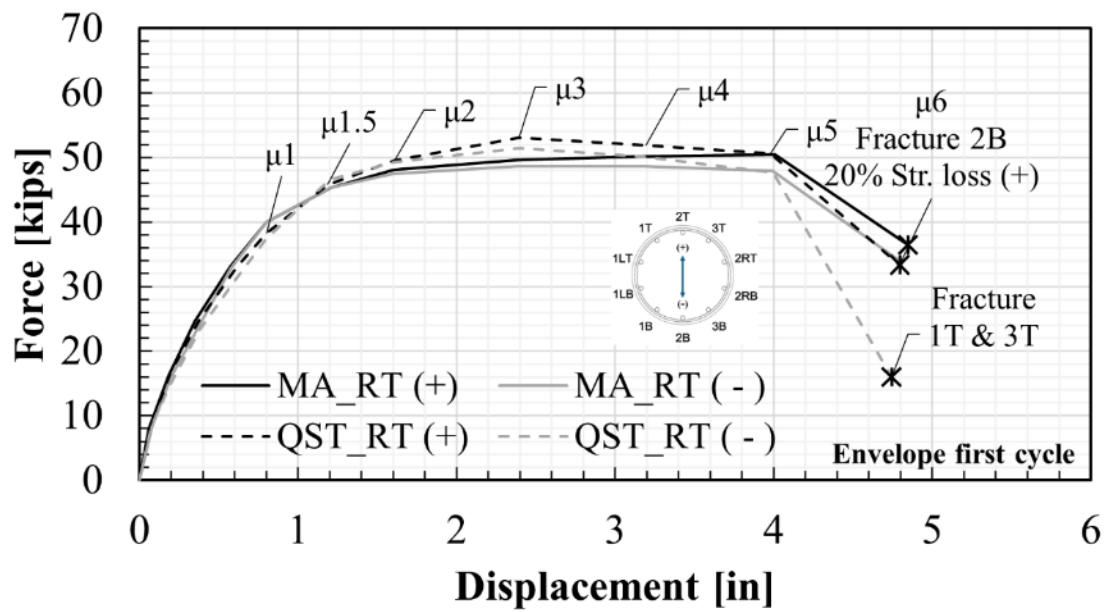


Figure 5-146 Comparison of the first cycle envelope of the force displacement response of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in.

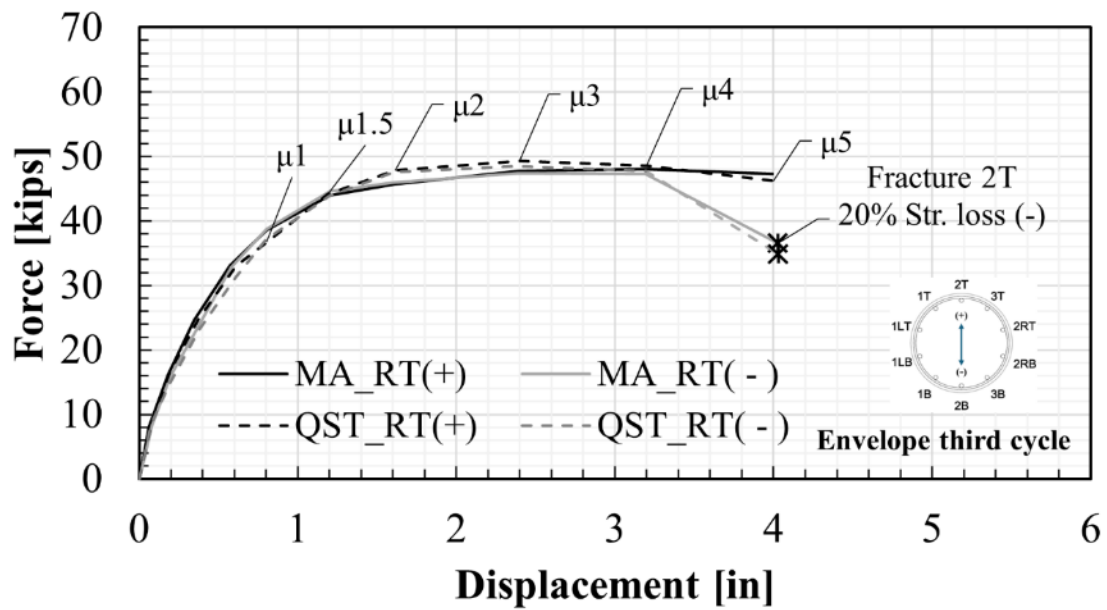


Figure 5-147 Comparison of the third cycle envelope of the force displacement response of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in.

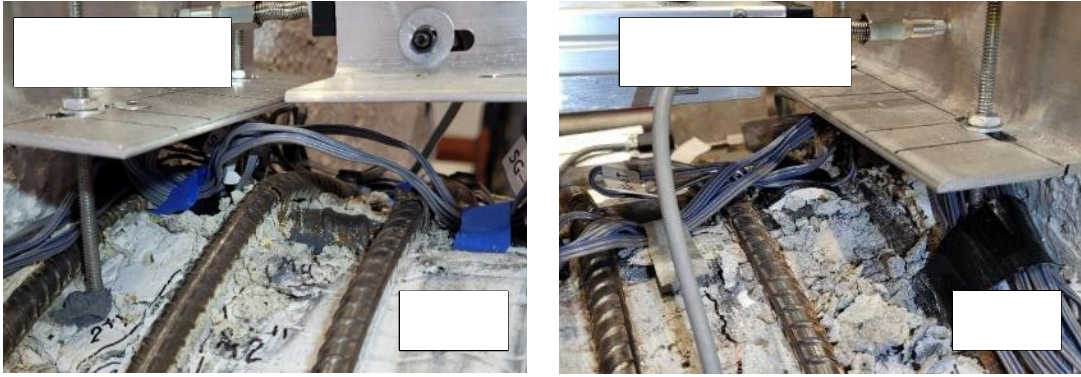


Figure 5-148 Comparison of the MA and QST columns tested at room temperature, $\mu 5^{+1}$: buckling in the MA specimen, and significant crushing along the top extreme bar in the QST specimen.

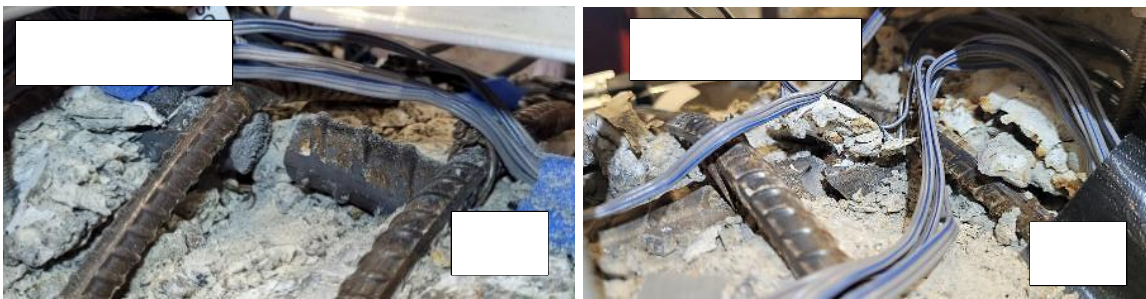


Figure 5-149 Comparison of the top bar fractures on the MA and QST columns tested at room temperature.

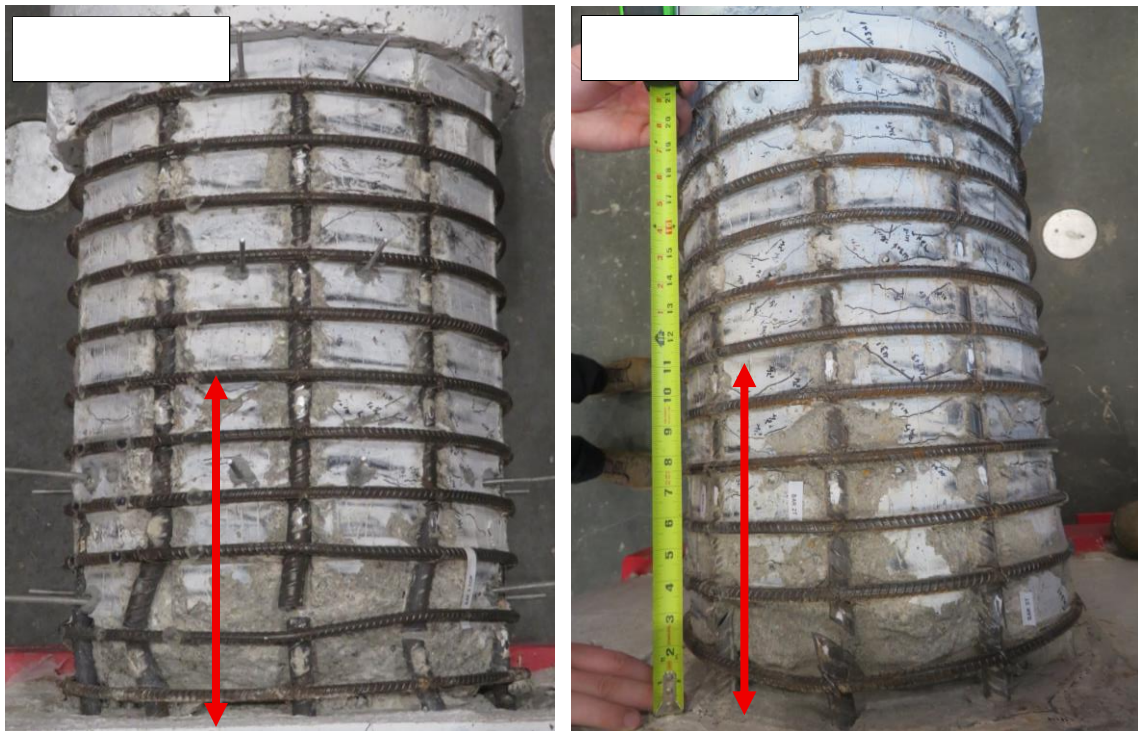


Figure 5-150 Concentration of concrete crushing at the base of the columns

The curvature distributions of Columns 1 and 2 for ductility 1 and 4 are shown in Figure 5-151 and Figure 5-152, respectively. The curvature profiles were comparable between the QST and the MA specimens tested at room temperature. The QST specimen showed a slightly larger spread of plasticity, while the MA specimen showed a higher concentration of curvature near the base of the column. These observations are consistent with the final condition of the columns displayed in Figure 5-150 where crushing extended over a larger distance in the QST specimen, whereas it was more severe and localized near the base of the column in the MA. However, these differences are minor, and the overall behavior of the column was similar for both specimens.

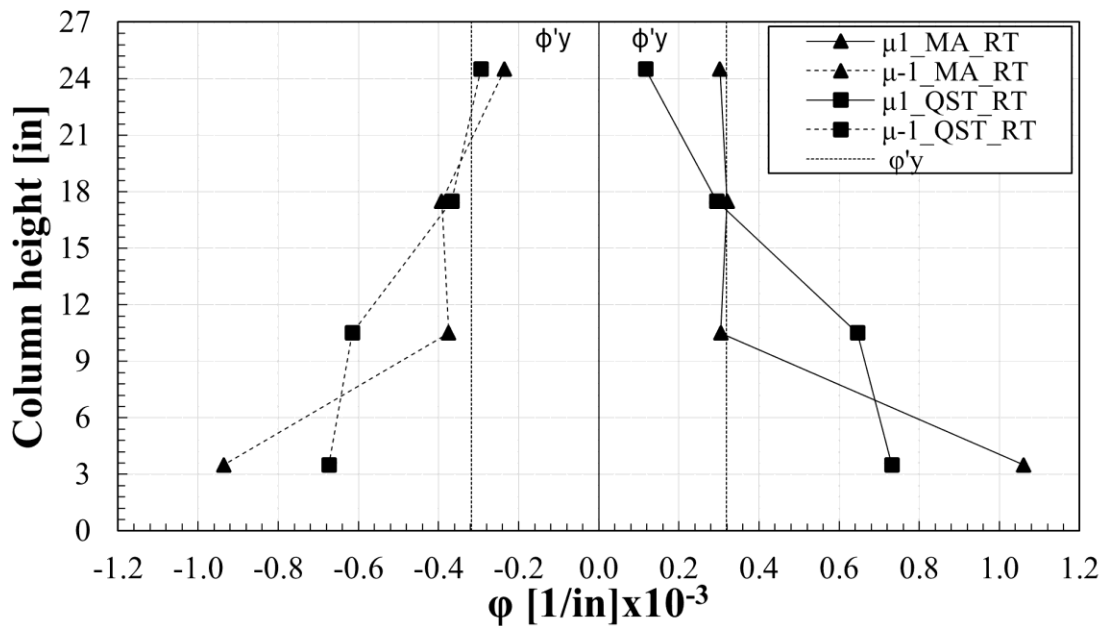


Figure 5-151 Comparison of the curvature distribution at μ_1 of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in

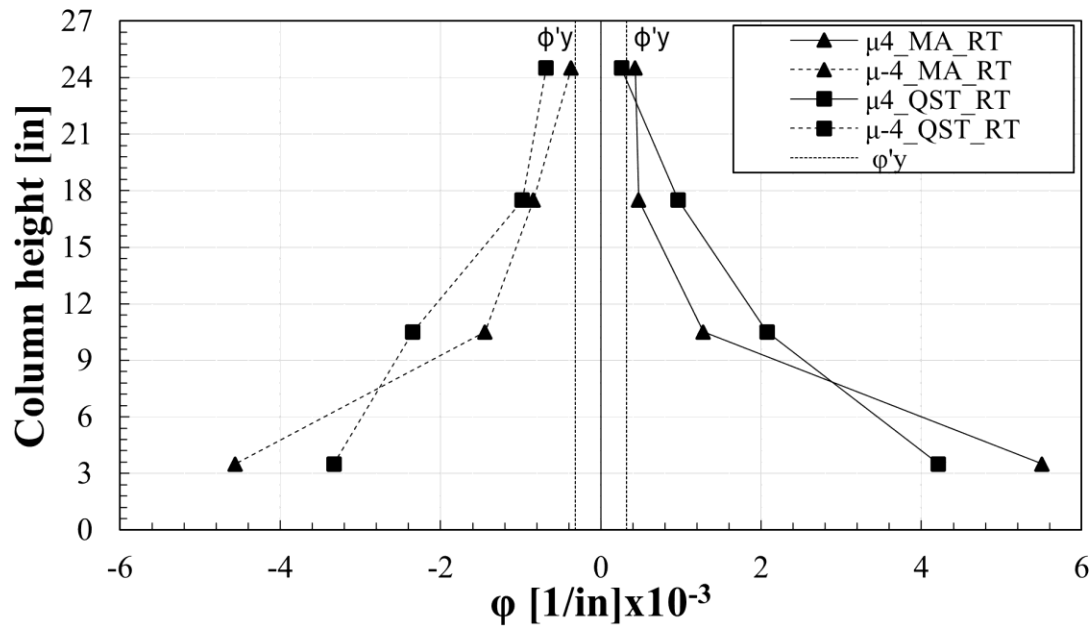


Figure 5-152 Comparison of the curvature distribution at μ_4 of Columns 1 and 2: MA_RT_2in vs. QST_RT_2in

5.10.3.2. Low temperature observations

Columns 3 and 4 corresponded to the specimen reinforced with MA and QST tested at low temperature. Figure 5-153 shows the comparison of the hysteretic response of the columns. The MA was loaded until the third cycle of μ_5 , while the QST column reached the third cycle of μ_5 . Both columns achieved similar maximum forces: 58.9 kips for the MA column and 57.8 kips for the QST, which were 17% and 9% higher than those obtained from each specimen pair tested at room temperature.

When comparing the experimental responses of the MA and QST specimens tested at low temperatures with each room temperature pair, the impact of the low temperature appears consistent. Both low-temperature specimens exhibited similar damage progression, with a delay in the onset of concrete cracking, crushing, and bar buckling relative to the room-temperature tests. The delay in damage progression occurs due to the enhancement of concrete and steel strength caused by low temperatures. However, the QST specimen tested

at low temperature had a faster strength degradation than the MA specimen after the first bar buckled. This difference in strength degradation is shown in Figure 5-154 and Figure 5-155, which compare the first and third force-displacement response envelopes. In the force-displacement response envelopes, the QST specimen strength deteriorated by approximately 60% by the end of the third cycle of μ_5 , after four bar fractures, and the test was terminated. In contrast, the MA specimen showed greater residual strength at the end of μ_5 and was loaded until the third cycle of μ_6 , at which point about 50% strength degradation was observed.

The MA and QST columns tested at low temperature also showed indications of brittle bar fractures (Figure 5-157). In both cases, the bar fractures occurred in the subsequent half-cycle after visible buckling (Figure 5-156). At the same time, the room-temperature columns withstand approximately three complete cycles between buckling and bar fracture.

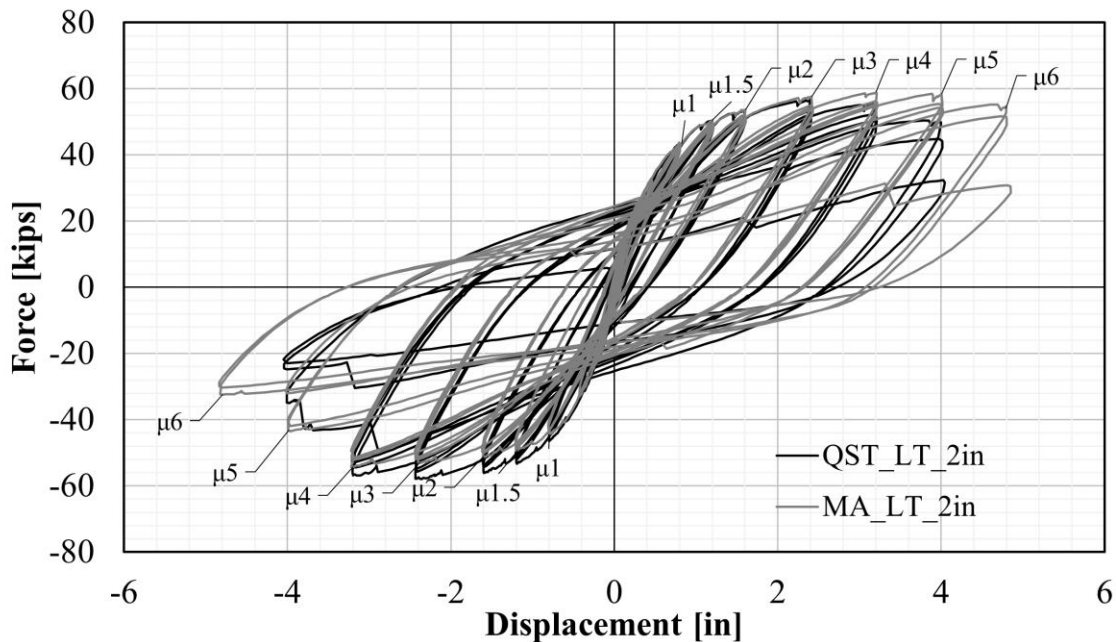


Figure 5-153 Comparison of the hysteretic response of Columns 3 and 4: MA_LT_2in vs. QST_LT_2in.

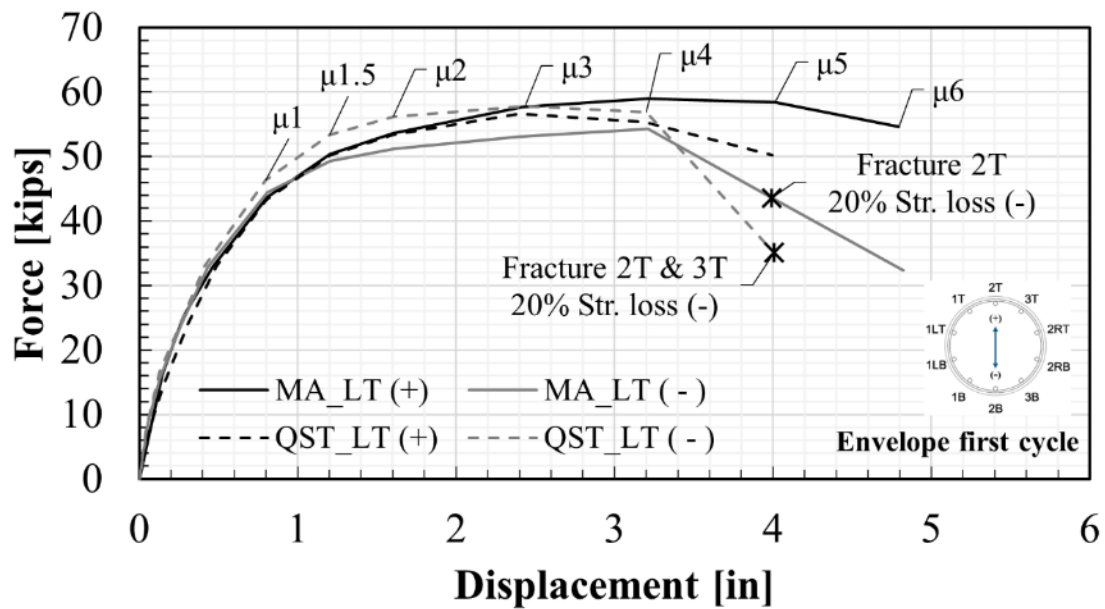


Figure 5-154 Comparison of the first cycle envelope of the force displacement response of Columns 3 and 4:
MA_LT_2in vs. QST_LT_2in

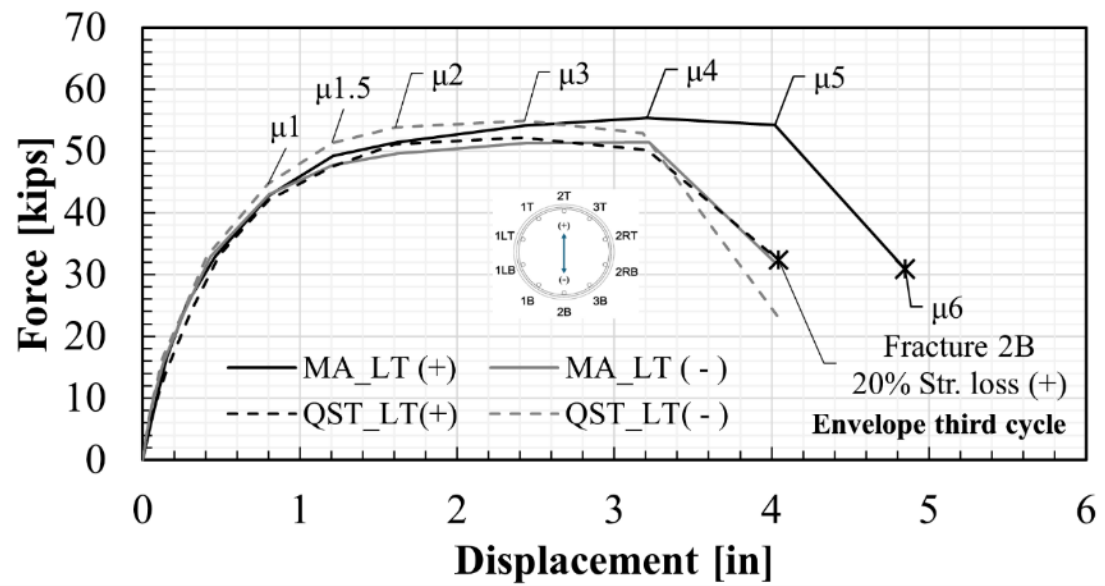


Figure 5-155 Comparison of the third cycle envelope of the force displacement response of Columns 3 and 4:
MA_LT_2in vs. QST_LT_2in



Figure 5-156 Top extreme bar buckling for the MA and QST columns with 2-inch spiral spacing tested at low temperatures.



Figure 5-157 Top extreme bar fractures for the MA and QST columns with 2-inch spiral spacing tested at low temperatures

Figure 5-158 and Figure 5-159 present the curvature distributions of Columns 3 and 4 for ductility 1 and 4, respectively. At low temperatures, the MA and QST columns with a transverse steel spacing of 2 in presented a similar distribution of curvatures. At μ_1 , the MA specimen presented a higher concentration of curvature near the base of the column in the pushing direction (dashed lines); however, in the pulling direction (solid lines), the greater curvature values were concentrated between the first and second gages, similarly to the distribution obtained for the QST specimen. At μ_4 , both specimens had a symmetrical distribution, with larger curvatures concentrated near the base of the column. For both cases, the curvatures obtained in the pulling direction were higher than in the other direction, which aligns well with the experimental observations, where both columns presented bar fracture first at the top of the column. In addition, the QST exhibited greater curvature in the pulling direction than at room temperature.

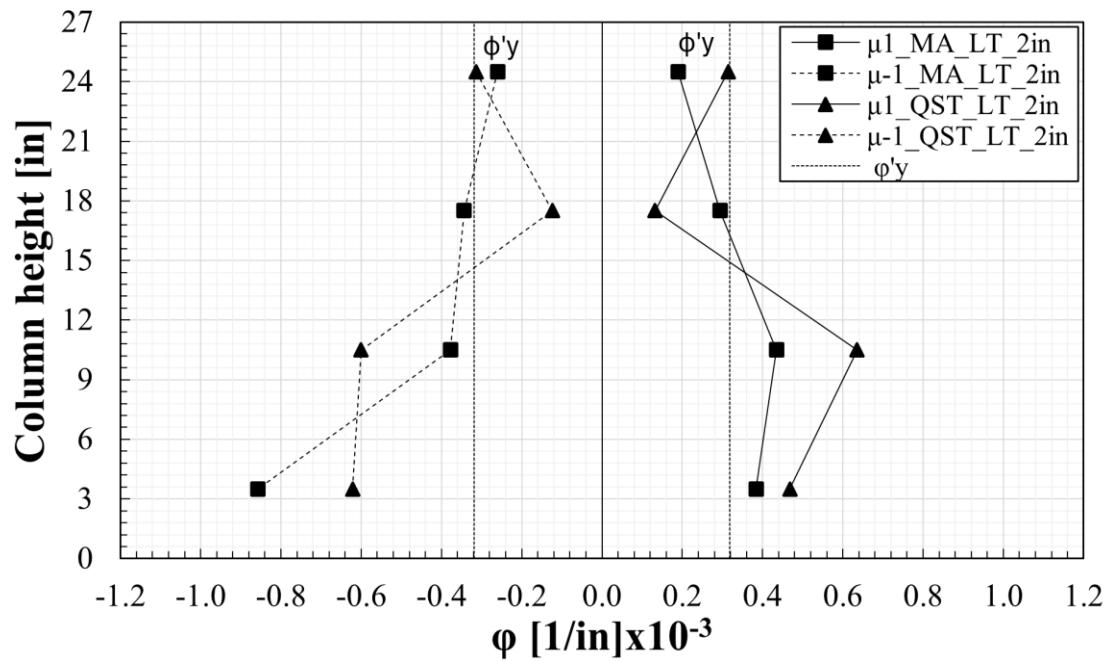


Figure 5-158 Comparison of the curvature distribution at μ_1 of Columns 3 and 4: MA_LT_2in vs. QST_LT_2in

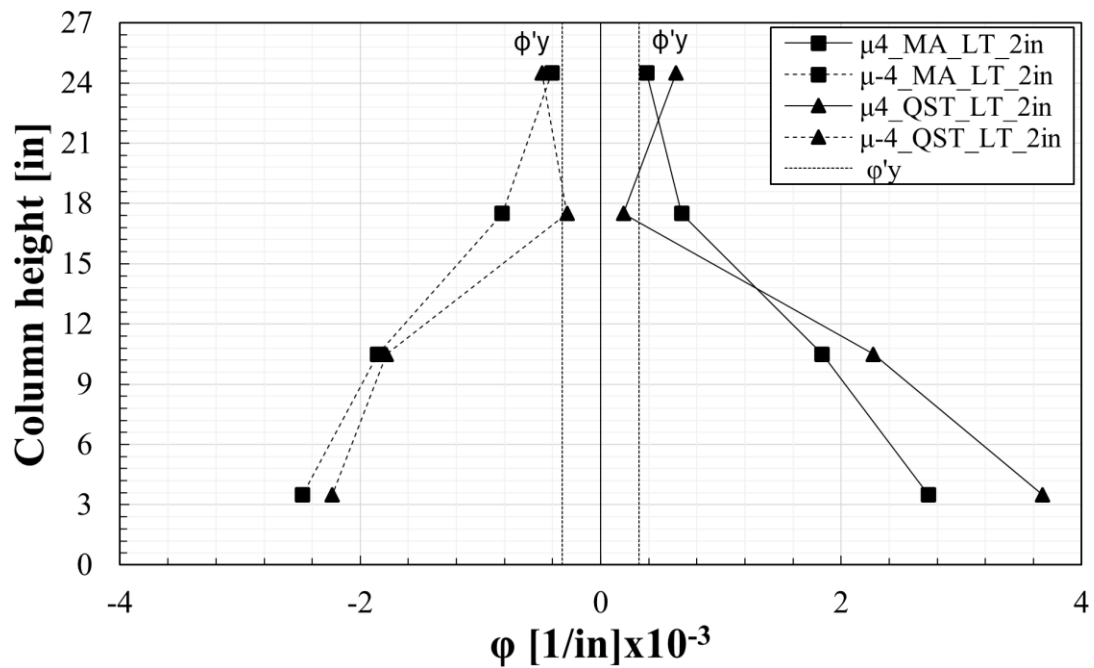


Figure 5-159 Comparison of the curvature distribution at μ_4 of Columns 3 and 4: MA_LT_2in vs. QST_LT_2in

Similarly, columns 5 and 6 corresponded to specimens reinforced with MA and QST, respectively, tested at low temperature, with a greater spiral separation of 4 inches. This pair of columns doesn't have a control test at room temperature, as the previous specimens (3 and 4), and therefore are just comparable to each other. The objective with this set of specimens was to determine whether there were any significant differences between the manufacturing processes, MA and QST, under aggravated buckling conditions. Figure 5-160 shows the comparison of the hysteretic response of the columns, which exhibit similar behavior. The MA was loaded until the third cycle of $\mu 5$, whereas the QST column was loaded until the second cycle of $\mu 5$. Both columns achieved similar maximum forces: 56.8 kips for the MA column and 60.4 kips for the QST, with the QST column being 6% stronger than the MA column. Similarly, both columns reached a maximum displacement of 4 inches, corresponding to $\mu 5$.

Both low-temperature specimens exhibited comparable damage progression, with strength degradation occurring slightly faster in the QST-reinforced column than in the MA-reinforced column. Figure 5-161 shows a 20% strength degradation in both loading directions for the QST column during the first pulling and pushing cycles of $\mu 5$. While the 20% strength degradation occurred in the second and third cycles of $\mu 5$ for the MA column (Figure 5-162).

As seen in the material-level evaluation, the failure mode of the reinforcing steel was more brittle at low temperatures. The fracture surfaces of the longitudinal bar showed a non-uniform failure surface (like a "stair step"), which differs from the most flat surface typically obtained on MA bars, and the QST steel showed a flat failure surface, where diagonal bar fractures tend to occur (Figure 5-165).

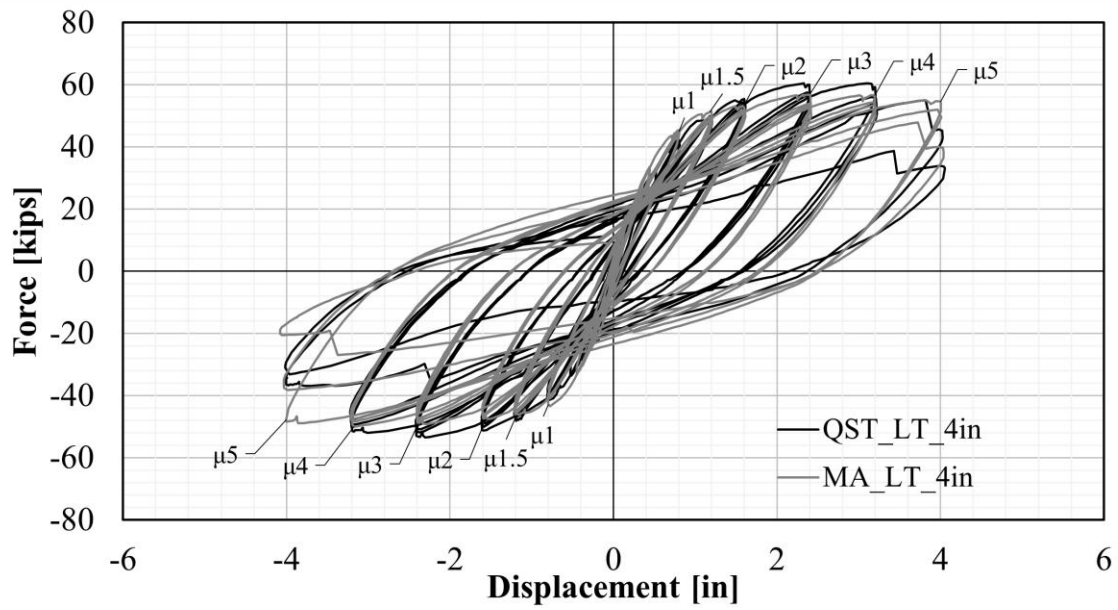


Figure 5-160 Comparison of the hysteretic response of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in.

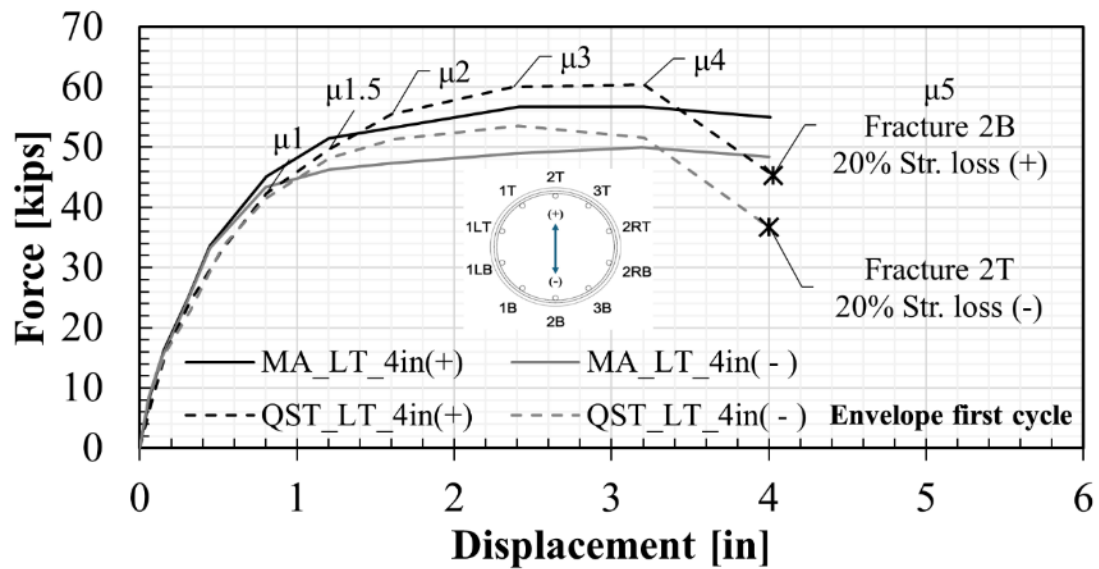


Figure 5-161 Comparison of the first cycle envelope of the force displacement response of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in

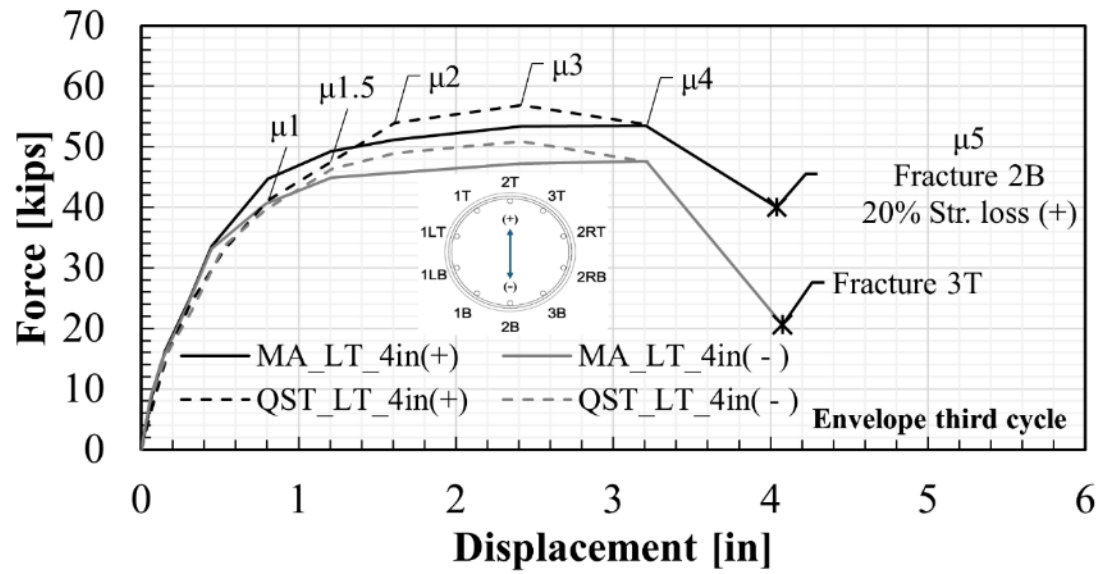


Figure 5-162 Comparison of the third cycle envelope of the force displacement response of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in

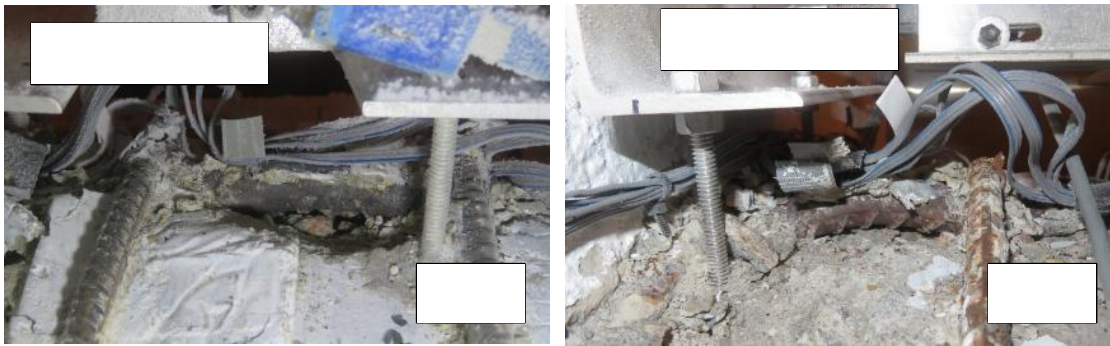


Figure 5-163 Buckling of the top extreme top bars of specimens 5 and 6

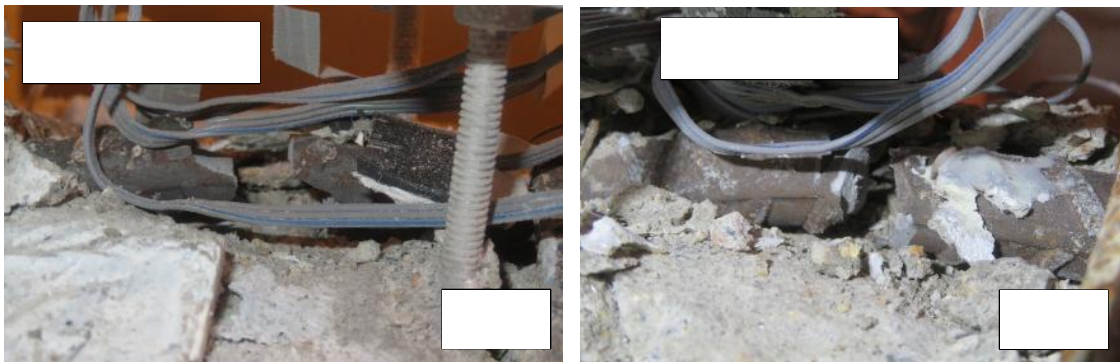


Figure 5-164 Fracture of the top extreme bars of specimens 5 and 6

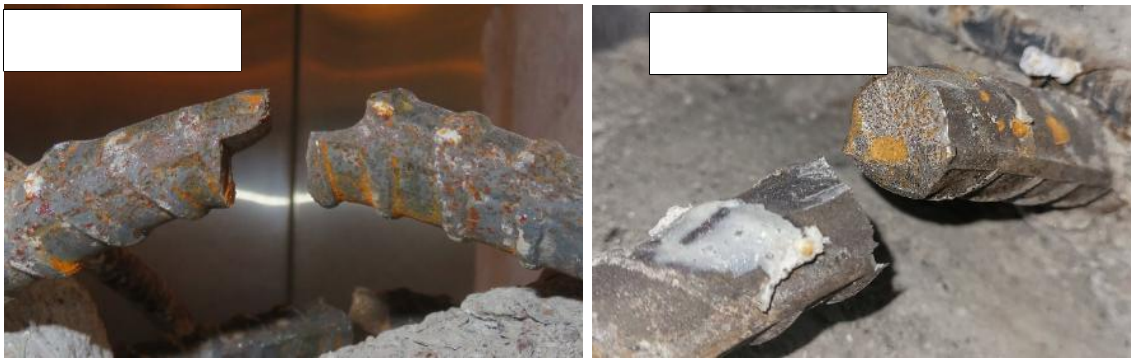


Figure 5-165 Brittle fracture of the MA and QST longitudinal steel.

Figure 5-158 and Figure 5-159 present the curvature distributions of Columns 5 and 6 for ductility 1 and 4, respectively. Similar to the observations on the low-temperature specimens with a 2 in spiral spacing, the MA and QST columns with a transverse steel spacing of 4 in presented a similar distribution of curvatures. At $\mu 1$, both columns showed a greater curvature in the third gage at a column height of approximately 17.5 in the pushing direction (dashed lines). While in the pulling direction (solid lines), the greater curvature values were concentrated near the column base for the MA specimen and at the third gage for the QST specimen. At $\mu 4$, both specimens exhibited different curvature distributions in the pushing and pulling directions. The QST column presented a larger curvature concentrated near the base of the column in the pushing direction, which correlated well with the bar fracture occurring first at the bottom extreme bar for this specimen. In contrast, the MA column presented a larger curvature concentrated near the base of the column in the pulling direction, correlating well with the bar fracture occurring first at the top extreme bar in this specimen.

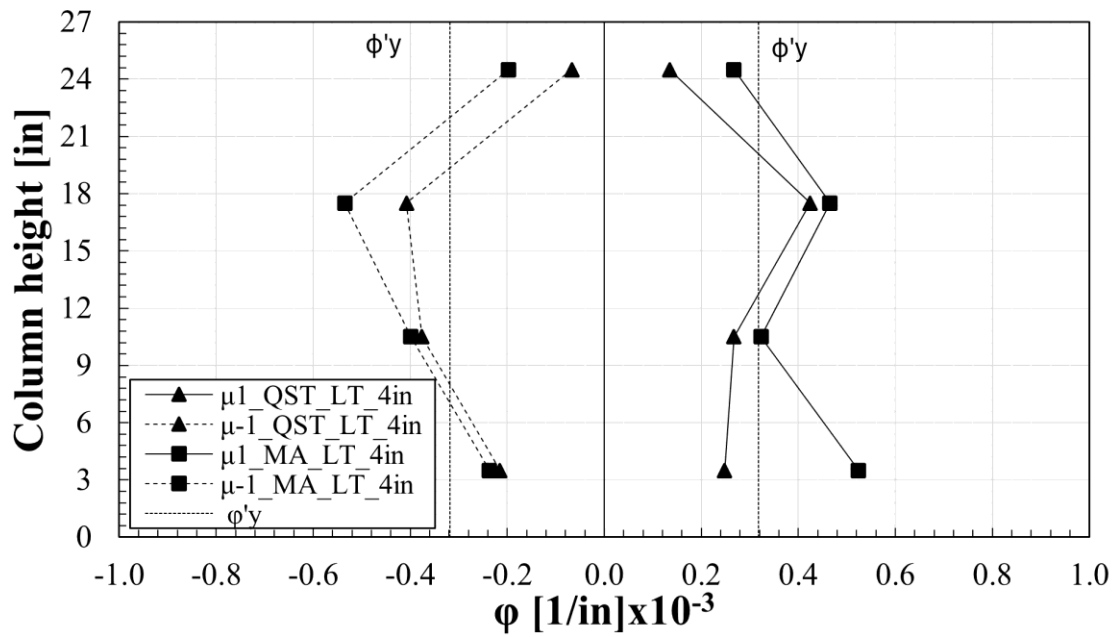


Figure 5-166 Comparison of the curvature distribution at μ_1 of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in

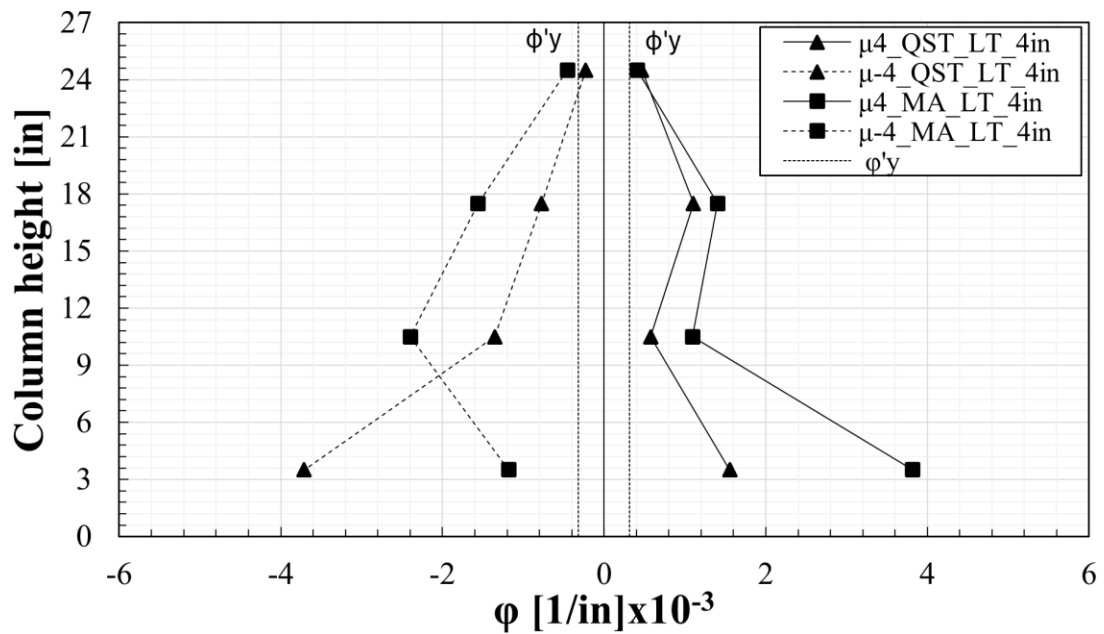


Figure 5-167 Comparison of the curvature distribution at μ_4 of Columns 5 and 6: MA_LT_4in vs. QST_LT_4in

5.11. Summary

Six columns reinforced with ASTM A706 grade 80 steel were subjected to quasistatic lateral loading to address their seismic behavior at room and low temperatures. All the columns had a diameter of 18 inches, a clear height of 65 inches, and were reinforced using 10 #6 grade 80 longitudinal bars. Two main steel manufacturing processes were studied, MA steel and QST steel. The first four columns had a spiral separation of 2 inches and corresponded to two pairs of identical columns that were tested at room and low temperatures using MA and QST steel. The first set of columns was tested to assess the effect of temperature on the MA and QST steels, as well as the differences between the two manufacturing processes at each temperature. The final set of columns corresponded to specimens built using MA and QST with a larger spiral separation of 4 in. This last set of columns was designed to address whether significant differences in behavior exist between MA and QST steel under aggravated buckling conditions. All the columns were subjected to the same loading protocol. A summary of the experimental observations is listed below:

5.11.1. Low temperature impact on the seismic behavior of HSRS

- The MA and QST specimens tested at low temperature showed an increase in strength of 17% and 9% relative to their room temperature counterparts.
- The global response of the low temperature specimens was comparable to the room temperature columns, despite differences in damage progression and reduction in the bending strain capacity of their reinforcing steel. The low temperatures increased the strength of concrete and transverse steel, delaying the onset of buckling and counteracting the reduction in bending strain capacity.

- The bar fracture of the low-temperature specimens appears to be more brittle in nature compared to the room-temperature specimens.
- Based on the curvature distribution profiles, low temperature did not significantly affect the spread of plasticity. Higher curvature values were observed near the base of the columns in the room-temperature specimens compared to the low-temperature specimens

5.11.2. Manufacturing process impact on the seismic behavior of HSRs at low temperatures

- MA and QST steel presented comparable behavior at room and low temperatures, in both confinement conditions (2 inches and 4 inches of spacing between spiral layers). The QST steel exhibited slightly higher strength in all comparisons and faster strength degradation than MA columns after the first bar fracture; however, displacement capacity was not affected. Usually, both manufacturing processes had the first bar fracture and 20% strength loss at the same ductility level in comparable conditions.
- The curvature distributions were comparable between the MA and QST specimens in all the evaluated cases. Minor differences were observed at room temperature, where the QST specimens appeared to exhibit a slightly higher spread of plasticity. In contrast, the MA specimens showed higher curvature concentrated at the base of the column. These differences were not observed at low temperatures, where the curvature distribution obtained from the MA and QST specimens was very similar for both pairs of specimens with 2 and 4 inches spiral separation.

- The moment curvature predictions aligned very well with the experimental responses at room and low temperatures, which indicates that the low temperature response can be predicted with a conventional approach by using the expected material properties at low temperature. Further research is needed to more accurately predict the strength degradation of the columns at both room and low temperatures, particularly regarding bar buckling and bar fractures.

CHAPTER 6. CONCLUSIONS AND DESIGN RECOMMENDATIONS

This chapter summarizes the main findings obtained from the experimental program to understand the seismic behavior of HSRS at low temperatures. The material-level conclusions and recommendations are presented first, followed by those obtained at the member level. This chapter concludes with a description of future work.

6.1. Conclusions

6.1.1. Material level conclusions

Chapter 4 presented the material level evaluation of HSRS at low temperatures. In this phase, tension and BBT tests were performed at various temperatures to assess the impact of low temperatures on mechanical properties and on the critical bending strain on HSRS. The main findings are summarized below:

- BBT tests performed on grade 80 steel at room and low temperatures suggested that the manufacturing process could have an impact on the reinforcing steel's ability to sustain buckling at low temperatures. From these tests, the critical bending strain of QST bars was almost unaffected by temperature. While in MA bars, a 20% reduction in ϵ_{bc} was observed as the temperature decreased to -40 °F (-40 °C). These observations suggested that MA bars are more likely to exhibit an earlier ductile-to-brittle transition when subjected to increases of bending strain due to buckling, in contrast to the QST bars.
- Smooth bars exhibited higher ϵ_{bc} at both low and room temperatures compared to both ribbed QST and MA bars. The absence of ribs in the bar (and therefore stress concentrations in the section of maximum curvature),

combined with temperature variations in the experimental setup, prevented conclusive results for this group of bars, for which the effect of stress concentrations was to be evaluated.

- Tension tests showed that low temperatures resulted in increases in yield and ultimate stress for grade 80 and grade 100 steel as the temperature decreased. The T/Y ratio and the elastic modulus remained mainly unaffected by temperature. In addition, no clear trend was found on the effect of low temperatures on uniform elongation.

6.1.2. Member-level conclusions

Chapter 5 presented the large-scale experimental program conducted on 6 RC columns reinforced with ASTM A706 grade 80 MA and QST steel to evaluate the impact of low temperatures on the seismic behavior of HSRS. The first four specimens share the same detailing, with the only difference being the manufacturing process for the longitudinal reinforcing steel. Each pair of columns (reinforced with MA or QST longitudinal steel) was evaluated at room and low temperatures to assess the impact of manufacturing on the low-temperature seismic behavior of RC columns. The last two specimens were evaluated at low temperature with increased transverse steel spacing to assess differences in the manufacturing process under aggravated buckling conditions. From the experimental observations, the following conclusions were made:

- The global response of the low-temperature specimens was comparable to the room-temperature columns. The low temperature increased lateral strength in both MA and QST specimens, which were 17% and 9% higher than those obtained from the corresponding specimen pairs tested at room temperature.

No significant variations were obtained in the displacement capacity of the columns tested at low temperatures.

- The MA and QST columns tested at low temperatures exhibited differences in the damage progression and reduction in the bending strain capacity of their reinforcing steel compared to their room temperature pairs. However, the increased strength of concrete and transverse steel caused by low temperatures delayed the onset of buckling. It counteracted the reduction in bending strain capacity, resulting in comparable global behavior.
- The curvature distribution profiles did not show a significant effect of the low temperatures on the spread of plasticity. However, higher curvature values were observed near the base of the columns in the room-temperature specimens compared to the low-temperature specimens.
- Regarding the potential impact of reinforcing steel manufacturing process, the MA and QST steel presented comparable behavior at room and low temperatures, in both confinement conditions (2-inch and 4-inch spiral spacing). The QST steel exhibited slightly higher strength in all comparisons and faster strength degradation than MA columns after the first bar fracture; however, displacement capacity was not affected. Usually, both manufacturing processes had the first bar fracture and 20% strength loss at the same ductility level in comparable conditions. This conclusion differs from the expectation observed in the material-level evaluation, which indicated that the temperature had a different effect for both manufacturing processes; this difference can be explained by the difference in the QST steel used in the member-level evaluation. The QST steel used in the material-level

review did not meet the T/Y ratio requirements specified in ASTM A706, whereas the one used for the member-level assessment did. This is the result of manufacturers fine-tuning their processes during this research project.

- The moment curvature predictions aligned very well with the experimental responses at room and low temperatures, which indicates that the low temperature response can be predicted by using the expected material properties at low temperatures. Further research is needed to more accurately predict the strength degradation of the columns at both room and low temperatures, particularly regarding bar buckling and bar fractures.

6.2. Recommendations

6.2.1. Material level recommendations

6.2.1.1. Reinforcing steel:

Chapter 4 showed the effect of the low temperature on the grade 80 and grade 100 steel. The current recommendations in the Alaska Bridges and Structures Manual (2017) indicate using a 10% increase in yield and ultimate strength for the expected material properties of Grade 60 steel at temperatures below -4°F (-20 °C). However, based on the results of the tension test on HSRS using three bars at each evaluated temperature between 72°F (22 °C) and -40°F (-40°C), the 10% increment overpredicts low-temperature strength. The factors presented below are proposed for grade 80 steel at temperatures below -4°F (-20 °C).

Table 6-1 Low temperature strength increase factors for grade 80 steel

Reinforcing steel grade	Properties	Low temperature-based modifications
Grade 80	f_{ye} = expected yield stress of steel	$f_{ye-cold} = 1.07 \times f_{ye}$
	f_{ue} = expected tensile strength of steel	$f_{ue-cold} = 1.06 \times f_{ue}$

6.2.1.2. Concrete:

The effect of low temperature on concrete strength was not directly addressed in this research, since the primary focus was to assess the impact of temperature on the HSRS. However, comparing the results from the column concrete cylinders tested at room and low temperatures, a 31% to 76% increase in compressive strength was observed at low temperature, with an average increase of 50%. The compressive strength values obtained at low temperature were used directly to derive analytical predictions for the tested columns at low temperature, which matched the columns' experimental responses well, underscoring the importance of accounting for the increase in concrete strength. Previous recommendations suggest considering a 40% increase in concrete strength, which is within the observed strength increase range in this research. It is recommended to use the expected material properties, considering the previously proposed 40% strength increment or any other model found in the literature, at the engineer's discretion.

6.2.2. Member level recommendations

Based on the experimental observations obtained from the material and member level evaluation, the following recommendations for the implementation of HSRS in bridge design in cold seismic regions are made:

6.2.2.1. Selection of reinforcing steel:

- ASTM A706 grade 80 steel showed promising seismic behavior at -40°F (-40°C), with comparable force displacement response to the one obtained at room temperature for columns with a longitudinal steel ratio of 1.74%, transverse steel ratio of 1.23% at a separation of 3db, and axial load ratio of 8%.

- For MA and QST grade 80 steel, meeting the requirements of ASTM A706-22, no significant differences in seismic behavior at room temperature and low temperature (72°F/22°C and -40°F/-40°C) were observed, even with the columns tested with a transverse steel spacing of 6db. This shows that the manufacturing process does not pose a concern for the use of HSRS in cold seismic regions with temperatures down to -40°F (-40°C).

6.2.2.2. *Special considerations:*

- The design of RC columns at low temperature should be done accounting for the expected material properties, with the low temperature overstrength factor for the concrete and reinforcing steel.
- The curvature profiles obtained at low temperatures in columns showed lower curvature values compared to the room temperature specimens. This can also be interpreted as a lower strain demand in the low-temperature columns than in the room-temperature specimens at comparable displacements. Despite the apparent reduction in strain demand, bar fracture and strength degradation in the low-temperature specimens occurred at the same lateral displacement as those tested at room temperature. This indicates more brittleness in the steel at low temperatures, which also aligned with the experimental observations; however, the effect of temperature on the other materials, i.e., concrete and transverse steel, helps delay the onset of buckling and concrete damage, thereby compensating for the local effects on the reinforcing steel. It is important to emphasize that the compensation of the low temperature effect on the longitudinal steel with the confinement provided by the increase in the

strength of the spiral yielding strength was observed for a well-confined column, with a transverse steel ratio of 1.23% and separation of 3db. It is recommended to provide a similar level of confinement.

6.3. Further work

- The experimental results of this research suggest promising seismic performance behavior of HSRS at low temperature for well-confined specimens. However, these observations indicate the importance of confinement in mitigating the low-temperature impact on the longitudinal steel, which contributes to its brittleness. Further research is needed to define acceptable confinement ratios and spacing to ensure comparable seismic behavior at low and room temperatures for RC columns under seismic demand using HSRS.
- Conventional sectional analysis approach showed to predict well the experimental response of the specimens when using the low temperature material properties. However, further investigation is needed to obtain low-temperature-based limit states and capture the expected damage progression of RC columns. Specifically, determine the strain demand in the onset of buckling and fracture at room and low temperatures and propose a temperature-based modified buckling limit state for grade 80 steel.
- Montejo et al. (2010) suggested a plastic hinge length reduction factor for RC columns reinforced with grade 60 steel to be used for RC columns exposed to temperatures below -4°F (-20 °C), based on experimental observations that showed a reduction in the spread of plasticity for the columns tested at low

temperature. However, no decrease in the extent of plasticity was observed in this research on columns reinforced with grade 80 steel at low temperatures. Plastic hinge length and the factors contributing to these differences should be further evaluated.

- At the material level, more BBT tests should be performed on the QST steel meeting ASTM A706 (with a higher T/Y ratio) to assess how the differences in the T/Y ratio on QST steel affect the critical bending strain at room and low temperatures.

REFERENCES

- AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition (2011).
- ACI Committee 318. (2011). *ACI CODE-318-11: Building Code Requirements for Structural Concrete and Commentary*. American Concrete Institute.
https://www.concrete.org/store/productdetail.aspx?ItemID=318U11&Language=English&Units=US_Units
- Alaska Bridges and Structures Manual (2017).
- ASTM A370-20. Standard Test Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370 (2020). <https://doi.org/10.1520/A0370-20>
- ASTM International. (2009). Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement. In *ASTM A706/A706M –09*.
https://compass.astm.org/document/?contentCode=ASTM%7CA0706_A0706M-09%7Cen-US
- ASTM International. (2022). *ASTM A706/706M-22a Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement*.
https://compass.astm.org/document/?contentCode=ASTM%7CA0706_A0706M-22A%7Cen-US
- Barbosa, A. R., Link, T., & Trejo, D. (2016). Seismic Performance of High-Strength Steel RC Bridge Columns. *Journal of Bridge Engineering*, 21(2), 04015044.
[https://doi.org/10.1061/\(asce\)be.1943-5592.0000769](https://doi.org/10.1061/(asce)be.1943-5592.0000769)
- Barclay, L., & Kowalsky, M. (2019a). Critical bending strain of reinforcing steel and the buckled bar tension test. *ACI Materials Journal*, 116(3), 53–61.
<https://doi.org/10.14359/51715583>
- Barclay, L., & Kowalsky, M. (2020). Seismic Performance of Circular Concrete Columns Reinforced with High-Strength Steel. *Journal of Structural Engineering*, 146(2), 04019198. [https://doi.org/10.1061/\(asce\)st.1943-541x.0002452](https://doi.org/10.1061/(asce)st.1943-541x.0002452)
- Barclay, L., Kowalsky, M. J., & North Carolina State University. Department of Civil, C. and E. E. (2019). *Seismic Behavior of Grade 80 RC Bridge Columns – Phase 1 and Critical Bending Strain of Longitudinal Reinforcement*.
<https://doi.org/10.21949/1503647>
- Elices, M., Planas, J., & Cortes, H. (1986). Behavior at Cryogenic Temperatures of Steel for Concrete Reinforcement. *JOURNAL OF THE AMERICAN CONCRETE INSTITUTE*, 83(3), 405–411. <https://www.webofscience.com/wos/woscc/full-record/WOS:A1986C687400008?UT=A1986C687400008&SID=USW2EC0D4DMFMsaJDg2EjHu2ZMX6H>

- Filiatrault, A., & Holleran, M. (2001). Stress-strain behavior of reinforcing steel and concrete under seismic strain rates and low temperatures. *Materials and Structures/Matériaux et Constructions*, 34, 235–239.
- Hyvonen, T. (1985). Brittleness and Strength of Reinforcing Steel under High Loading Rate at Lower Temperatures. *Nordic Concrete Research*, n 4, p 81-88.
https://www.engineeringvillage.com/share/document.url?mid=cpx_1907596&database=cpx&view=detailed
- ISO 6892-3:2015 - *Metallic materials — Tensile testing — Part 3: Method of test at low temperature*. (n.d.). Retrieved March 13, 2025, from
<https://www.iso.org/standard/51532.html>
- Kunitake, T., Hirakawa, K., Aihara, K., & Ohno, T. (1979). Concrete Reinforcement Steel Bars for Low-Temperature Services. *Sumitomo Search*, 22, 145–155.
https://www.engineeringvillage.com/share/document.url?mid=cpx_981642&database=cpx&view=detailed
- Levings, J., & Sritharan, S. (2012). Effects of Cold Temperature and Strain Rate on the Stress-Strain Behavior of ASTM A706 Grade 420(60) Steel Reinforcement. *Journal of Materials in Civil Engineering*, 24(12), 1441–1449.
[https://doi.org/10.1061/\(asce\)mt.1943-5533.0000523](https://doi.org/10.1061/(asce)mt.1943-5533.0000523)
- Li, Y., Cao, S., & Jing, D. (2018). Concrete columns reinforced with high-strength steel subjected to reversed cycle loading. *ACI Structural Journal*, 115(4), 1037–1048.
<https://doi.org/10.14359/51701296>
- Liu, X. yue, Wang, Y. qing, Zong, L., Lin, Y., & Shi, Y. jiu. (2014). Experimental study on mechanical properties and toughness of Q460C high-strength steel and its butt welded joint at low temperature. *International Journal of Steel Structures* 2014 14:3, 14(3), 457–469. <https://doi.org/10.1007/S13296-014-3003-Z>
- Manhard, R., & Kowalsky, M. (2019). *Impact of Grade 80 Reinforcing Steel Production Process on the Seismic Behavior of Bridge Columns*. North Carolina State University.
- Montejo, L. A. (2007). *CUMBIA-Set of codes for the analysis of reinforced concrete members Set of Codes for the Analysis of Reinforced Concrete Members Theory and User Guide*. www.geocities.com
- Montejo, L. A., Sloan, J. E., Kowalsky, M. J., & Hassan, T. (2008). Cyclic Response of Reinforced Concrete Members at Low Temperatures. *Journal of Cold Regions Engineering*, 22(3), 79–102. [https://doi.org/10.1061/\(ASCE\)0887-381X\(2008\)22:3\(79\)/ASSET/35D8F010-2A5E-46F0-9748-F4FE53958D4A/ASSETS/IMAGES/LARGE/26.JPG](https://doi.org/10.1061/(ASCE)0887-381X(2008)22:3(79)/ASSET/35D8F010-2A5E-46F0-9748-F4FE53958D4A/ASSETS/IMAGES/LARGE/26.JPG)
- Montejo, L., Kowalsky, M. J., & Hassan, T. (2009a). Seismic Behavior of Flexural Dominated Reinforced Concrete Bridge Columns at Low Temperatures. *Journal of*

Cold Regions Engineering, 23(1), 18–42. [https://doi.org/10.1061/\(ASCE\)0887-381X\(2009\)23:1\(18\)](https://doi.org/10.1061/(ASCE)0887-381X(2009)23:1(18))

- Montejo, L., Kowalsky, M. J., & Hassan, T. (2009b). Seismic Behavior of Shear-Dominated Reinforced Concrete Columns at Low Temperatures. *ACI Structural Journal*, 106(4).
- Montejo, L., Sloan, J. E., Kowalsky, M. J., & Hassan, T. (2008). Cyclic Response of Reinforced Concrete Members at Low Temperatures. *Journal of Cold Regions Engineering*, 22(3), 79–102. [https://doi.org/10.1061/\(asce\)0887-381x\(2008\)22:3\(79\)](https://doi.org/10.1061/(asce)0887-381x(2008)22:3(79))
- Mori, T., Harada, T., Ooba, T., Ueno, T., Nishino, J., & Sakaguchi, M. (1984). Development of High-Strength Reinforcing Bar for Low-Temperature Service, by means of controlled rolling and Surface Quenching. *Transactions of the Iron and Steel Institute of Japan*, v 25, n 3, p b. 103.
https://www.engineeringvillage.com/share/document.url?mid=cpx_1817266&database=cpx&view=detailed
- Morimoto, H., Nishida, K., Kamada, Y., Ohtani, H., & Nakasato, F. (1982). Thermo-mechanical Treatment of Steel Bars. (Strength-toughness relationship of Controlled Rolled Reinforcing Steel Bars -II). *Transactions of the Iron and Steel Institute of Japan*, v 23, n 4, p b-135.
https://www.engineeringvillage.com/share/document.url?mid=cpx_1380738&database=cpx&view=detailed
- NIST. (2014). Use of High-Strength Reinforcement in Earthquake-Resistant Concrete Structures. NIST GCR 14-917-30 Report, prepared by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology. In *NITS: Vol. NIST GCR 14-917-30*.
- Otálora, L. M. E., & Kowalsky, M. (2022). *LITERATURE REVIEW ON SEISMIC BEHAVIOR OF HIGH STRENGTH REINFORCING STEEL AT LOW TEMPERATURES*. www.cfl.ncsu.edu
- Perez-Martin, M. J., Holmen, J. K., Thomesen, S., Hopperstad, O. S., & Børvik, T. (2019). Dynamic Behaviour of a High-Strength Structural Steel at Low Temperatures. *Journal of Dynamic Behavior of Materials*, 5(3), 241–250. <https://doi.org/10.1007/S40870-019-00206-X/FIGURES/11>
- Priestley, M. J. N., Seible, F., & Calvi, G. M. (1996). Seismic Design and Retrofit of Bridges. *Seismic Design and Retrofit of Bridges*.
<https://doi.org/10.1002/9780470172858>
- Rautenberg, J. M., Pujol, S., Tavallali, H., & Lepage, A. (2012). Reconsidering the use of high-strength reinforcement in concrete columns. *Engineering Structures*, 37, 135–142. <https://doi.org/10.1016/J.ENGSTRUCT.2011.12.036>

- Restrepo-Posada, J. I., Dodd, L. L., Park, R., & Cooke, N. (1994). Variables Affecting Cyclic Behavior of Reinforcing Steel. *Journal of Structural Engineering*, 120(11), 3178–3196. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1994\)120:11\(3178\)](https://doi.org/10.1061/(ASCE)0733-9445(1994)120:11(3178))
- Rodriguez-Martin, R., Ocana, I., & Martin-Meizoso, A. (2007). Brittle fracture of high-strength steels at very low temperatures. *Fatigue & Fracture of Engineering Materials & Structures*, 30(7), 664–670. <https://doi.org/10.1111/J.1460-2695.2007.01131.X>
- Seismic Design Criteria Version 2.0, State of California Department on Transportation (2019).
- Sloan, J. E., Kowalsky, M. J., & Hassan, T. (2005). *The Seismic Behavior of Reinforced Concrete Members at Low Temperatures*. North Carolina State University.
- Sokoli, D., & Ghannoum, W. M. (2016). High-strength reinforcement in columns under high shear stresses. *ACI Structural Journal*, 113(3), 605–614. <https://doi.org/10.14359/51688203>
- Sokoli, D., Limantono, A., & Ghannoum, W. M. (2020). Special Moment Frames with High-Strength Reinforcement—Part 2: Columns. *Structural Journal*, 117(2), 253–265. <https://doi.org/10.14359/51721319>
- Sritharan, S., Eeri, M., Suleiman, M. T., & White, D. J. (2007). *Effects of Seasonal Freezing on Bridge Column-Foundation-Soil Interaction and Their Implications*. <https://doi.org/10.1193/1.2423071>
- Thangjitham, J. S. (2023). *Seismic Design Recommendations for Grade 80 Reinforcing Steel in Concrete Bridge Columns*. <https://repository.lib.ncsu.edu/handle/1840.20/40709>
- Tong, L., Niu, L., Jing, S., Ai, L., & Zhao, X. L. (2018). Low temperature impact toughness of high strength structural steel. *Thin-Walled Structures*, 132, 410–420. <https://doi.org/10.1016/J.TWS.2018.09.009>
- Trejo, D., Barbosa, A. R., Link, T., & University, O. S. (2014). *Seismic performance of circular reinforced concrete bridge columns constructed with grade 80 reinforcement*. <https://doi.org/10.21949/1503647>
- UK CARES. (2011). *The CARES Guide to Reinforcing Steels Manufacturing Process Routes for Reinforcing Steels*.
- Xie, J., Zhao, X., & Yan, J. B. (2018). Mechanical properties of high strength steel strand at low temperatures: Tests and analysis. *Construction and Building Materials*, 189, 1076–1092. <https://doi.org/10.1016/J.CONBUILDMAT.2018.09.053>
- Yan, J. B., Liew, J. Y. R., Zhang, M. H., & Wang, J. Y. (2014). Mechanical properties of normal strength mild steel and high strength steel S690 in low temperature relevant to Arctic environment. *Materials & Design*, 61, 150–159. <https://doi.org/10.1016/J.MATDES.2014.04.057>

- Yan, J. B., Luo, Y. L., Lin, X., Luo, Y. B., & Zhang, L. (2021). Effects of the Arctic low temperature on mechanical properties of Q690 and Q960 high-strength steels. *Construction and Building Materials*, 300. <https://doi.org/10.1016/J.CONBUILDMAT.2021.124022>
- Yan, J. B., & Xie, J. (2017). Experimental studies on mechanical properties of steel reinforcements under cryogenic temperatures. *Construction and Building Materials*, 151, 661–672. <https://doi.org/10.1016/J.CONBUILDMAT.2017.06.123>