

1. Report No. FHWA/TX-09/0-4588-1 Vol. 1		2. Government Accession No.		3. Recipient's Catalog No.	
4. Title and Subtitle EFFECT OF VOIDS IN GROUTED, POST-TENSIONED CONCRETE BRIDGE CONSTRUCTION: VOLUME 1 – ELECTROCHEMICAL TESTING AND RELIABILITY ASSESSMENT				5. Report Date February 2009 Published: September 2009	
				6. Performing Organization Code	
7. Author(s) David Trejo, Mary Beth D. Hueste, Paolo Gardoni, Radhakrishna G. Pillai, Kenneth Reinschmidt, Seok Been Im, Suresh Kataria, Stefan Hurlebaus, Michael Gamble, and Thanh Tat Ngo				8. Performing Organization Report No. 0-4588-1 Vol. 1	
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135				10. Work Unit No. (TRAIS)	
				11. Contract or Grant No. Project 0-4588	
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P. O. Box 5080 Austin, Texas 78763-5080				13. Type of Report and Period Covered Technical Report: September 2003 – August 2008	
				14. Sponsoring Agency Code	
15. Supplementary Notes Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration. Project Title: Effect of Voids In Grouted, Post-Tensioned Concrete Bridge Construction URL: http://tti.tamu.edu/documents/0-4588-1-Vol1.pdf					
16. Abstract Post-tensioned (PT) bridges are major structures that carry significant traffic. PT bridges are economical for spanning long distances. In Texas, there are several signature PT bridges. In the late 1990s and early 2000s, several state highway agencies identified challenges with the PT structures, mainly corrosion of the PT strands. The Texas Department of Transportation (TxDOT) performed some comprehensive inspections of its PT bridges. A consultant's report recommended that all ducts be re-grouted. However, the environment in Texas is very different than the environments in which the corrosion of the PT strands were observed. The objective of this research was to evaluate the corrosion activity of strands for PT structures and to correlate this corrosion activity with general environmental and void conditions. To achieve this objective, time-variant probabilistic models were developed to predict the tension capacity of PT strands subjected to different environmental and void conditions. Using these probabilistic models, time-variant structural reliability models were developed. The probability of failure of a simplified PT structure subjected to HS20 and HL93 loading conditions was assessed. Both flexural failure and serviceability were assessed. Results indicate that the presence of water and chlorides can lead to significant corrosion rates and failure is dependent on this corrosion activity and the number of strands exposed to these conditions. Volume 1 of this report presents these results. To assist TxDOT with developing a plan to mitigate this corrosion, studies were performed to assess repair grout materials, inspection methods, and repair methods. In addition, a general methodology is presented on optimizing repairs. These topics are presented in Volume 2 of this report. An Inspection and Repair Manual was also developed from this research and is presented in a separate report. Results indicate that TxDOT should prevent water and chlorides from infiltrating the tendons; this can be achieved in part by repairing drain lines and ducts and protecting anchor heads, as these conditions can lead to early failure of PT bridges. Recommendations on inspections, repairs, and materials are provided; however, further research on the potential formation of galvanic coupling of strands embedded in both existing and new repair grouts needs to be assessed.					
17. Key Words Post-tensioned Bridge; Corrosion; Electrochemistry, Strand, Reliability; Voids; Grout; Durability; Strength Reliability; Service Reliability; Deterioration			18. Distribution Statement No restrictions. This document is available to the public through NTIS: National Technical Information Service Springfield, Virginia 22161 http://www.ntis.gov		
19. Security Classif.(of this report) Unclassified		20. Security Classif.(of this page) Unclassified		21. No. of Pages 366	22. Price

**EFFECT OF VOIDS IN GROUTED, POST-TENSIONED
CONCRETE BRIDGE CONSTRUCTION: VOLUME 1 –
ELECTROCHEMICAL TESTING AND RELIABILITY
ASSESSMENT**

by

David Trejo, Ph.D., P.E., Associate Research Engineer
Mary Beth D. Hueste, Ph.D., P.E., Associate Research Engineer
Paolo Gardoni, Ph.D., Assistant Research Engineer
Radhakrishna G. Pillai, Graduate Student Researcher
Kenneth Reinschmidt, Ph.D., P.E., Research Engineer
Seok Been Im, Graduate Student Researcher
Suresh Kataria, Graduate Student Researcher
Stefan Hurlbaeus, Dr. Ing., Assistant Research Engineer
Michael Gamble, Graduate Student Researcher
Thanh Tat Ngo, Graduate Student Researcher

Zachry Department of Civil Engineering and
Texas Transportation Institute

Report 0-4588-1 Vol. 1

Project 0-4588

Project Title: Effect of Voids in Grouted, Post-Tensioned Concrete Bridge Construction

Performed in cooperation with the
Texas Department of Transportation
and the
Federal Highway Administration

February 2009

Published: September 2009

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration (FHWA) or the Texas Department of Transportation (TxDOT). References to specific products are for information only and do not imply any claim of performance for that particular product. This report does not constitute a standard, specification, or regulation. The researcher in charge was David Trejo, P.E. #93490.

ACKNOWLEDGMENTS

This project was conducted at Texas A&M University and was supported by TxDOT and FHWA through the Texas Transportation Institute (TTI). This project had several advisors from TxDOT, and their assistance and valuable input were very much appreciated. These engineers included Randy Cox (program coordinator), Jaime Sanchez (first project director), Maxine Jacoby (second project director), German Claros (third project director), and the following project advisors: Brian Merrill (Bridge Division), Kenny Ozuna (Bridge Division, Houston), Tom Rummel (Bridge Division), Dean Van Landuyt (Bridge Division), Kieth Ramsey, Gilbert Silva, and Steve Strmiska (Bridge Division). The authors also wish to thank Matt Potter of the High Bay Structural Materials Laboratory; Scott Crauneur of the Zachry Depart of Civil Engineering; Jeff Perry, Duane Wagner, Cheryl Burt, Scott Dobrovolny, Robert Kocman, and Gary Gerke of TTI; Daren Cline of the Department of Statistics at TAMU; Ceki Halmen, Ramesh Kumar, Byoung Chan, Rhett Dotson, and Laura Bolduc (former and current graduate students at TAMU); and the many people at TxDOT who assisted with the bridge inspections.

TABLE OF CONTENTS

Page

LIST OF FIGURES xvi

LIST OF TABLES xxii

EXECUTIVE SUMMARY 1

1. INTRODUCTION..... 3

1.1. PRESTRESSED CONCRETE TECHNOLOGY 3
1.2. GROUTED, POST-TENSIONED, SEGMENTAL CONCRETE BRIDGES 3
1.3. DEFINITIONS 3
1.4. CLASSIFICATION OF POST-TENSIONED SYSTEMS 4
 1.4.1. Internal post-tensioned systems 5
 1.4.2. External post-tensioned systems 6
1.5. RESEARCH MOTIVATION 6
1.6. RESEARCH OBJECTIVES 8
1.7. RESEARCH ASSUMPTIONS 9
1.8. RESEARCH METHODOLOGY 10
1.9. ORGANIZATION OF THE REPORT 11
1.10. ORGANIZATION OF VOLUME 1 12

2. LITERATURE REVIEW 15

2.1. INTRODUCTION 15
2.2. DETERIORATION OF POST-TENSIONED BRIDGES 15
 2.2.1. Failures of internal, grouted, post-tensioned systems 15
 2.2.1.1. *Bickton Meadows Footbridge, UK* 15
 2.2.1.2. *Ynys-y-Gwas Bridge, West Glamorgan, UK* 16
 2.2.2. Failures of external, grouted, post-tensioned systems 17
 2.2.2.1. *Niles Channel Bridge, Florida, US* 17
 2.2.2.2. *Mid-Bay Bridge, Florida, US* 18
 2.2.2.3. *Bob Graham Sunshine Skyway Bridge, Florida, US* 20
 2.2.2.4. *Varina-Enon Bridge, Virginia, US* 21

2.3.	PARAMETERS INFLUENCING CORROSION OF POST-TENSIONED SYSTEMS	23
2.3.1.	Dissimilar metallic materials in post-tensioned systems	23
2.3.2.	Grout class	24
2.3.3.	Voids	25
2.3.4.	Oxygen concentration	27
2.3.5.	Cementitious pore solution and pH.....	27
2.3.6.	Carbon dioxide concentration	29
2.3.7.	Moisture conditions and precipitation	30
2.3.8.	Time of wetness	32
2.3.9.	Relative humidity.....	32
2.3.10.	Chloride concentration.....	33
2.3.11.	Temperature	34
2.3.12.	Axial stress.....	34
2.3.13.	Other factors.....	35
2.4.	CORROSION EVALUATION USING VISUAL INSPECTION TOOLS.....	35
2.5.	MODELING CORROSION OF STEEL UNDER IMMERSION CONDITIONS	36
2.6.	MODELING CORROSION OF STEEL UNDER CONTINUOUS-ATMOSPHERIC EXPOSURE CONDITIONS	38
2.6.1.	Atmospheric corrosion models based on environmental parameters, time functions, or both	38
2.6.2.	Atmospheric corrosion models based on time functions only	40
2.6.3.	Developing and updating the power corrosion model	44
2.7.	STRUCTURAL CAPACITY OF POST-TENSIONED, SEGMENTAL BRIDGES	45
2.7.1.	Parameters directly influencing structural behavior of PT girders	45
2.7.1.1.	<i>Tension capacity of prestressing strands</i>	46
2.7.1.2.	<i>Axial stresses and their losses on strands</i>	47
2.7.1.3.	<i>Compressive strength of concrete</i>	49
2.7.2.	A history of design codes/practices for concrete bridges	49
2.7.3.	Differences in the behavior of monolithic and segmental, post-tensioned beams	50
2.7.4.	Stress-strain relationship for unbonded tendons	50
2.7.5.	Stress-strain relationship for concrete cross-sections	51
2.7.6.	In-service and allowable stresses in a cross-section	52
2.7.7.	Existing approaches to determine flexural capacity of prestressed girders	53
2.8.	STRUCTURAL DEMAND ON POST-TENSIONED BRIDGES	54
2.8.1.	Dead load	54
2.8.2.	Live and impact loads	55
2.9.	STRUCTURAL RELIABILITY OF POST-TENSIONED BRIDGES	56
2.9.1.	Structural reliability of bridges with uncorroded strands	57
2.9.2.	Modeling and assessment of structural reliability	58
2.9.2.1.	<i>Limit state functions</i>	58
2.9.2.2.	<i>Generalized structural reliability index and probability of failure</i>	59

2.9.2.3. <i>Assessing structural reliability using existing computer programs</i>	60
2.9.3. Target reliability index.....	61
2.10. TYPICAL CHARACTERISTICS OF SEGMENTAL CONCRETE BRIDGES IN TEXAS.....	63
2.10.1. Segmental concrete bridges in Texas.....	63
2.10.2. San Antonio “Y” bridge.....	64
2.10.3. Span and girder inventory in San Antonio “Y” bridge	67
2.10.4. Tendon inventory on San Antonio “Y” bridge	70
2.10.5. Cross-sectional properties of girders in San Antonio “Y” bridge.....	71
2.11. SUMMARY.....	73
3. CURRENT NEEDS AND RESEARCH SIGNIFICANCE.....	75
4. ENVIRONMENTAL CHARACTERIZATION MAPS OF TEXAS.....	79
4.1. INTRODUCTION	79
4.2. APPROACHES IN DEVELOPING MAPS.....	79
4.3. FREEZE-DAY, TEMPERATURE, RELATIVE HUMIDITY, AND RAIN-DAY MAPS	80
4.4. CHLORIDE MAPS.....	83
4.5. TOTAL CORROSION RISK MAPS	85
4.6. SUMMARY.....	87
5. EXPERIMENTAL PROGRAM – ELECTROCHEMICAL AND TENSION CAPACITY BEHAVIOR OF WIRES AND STRANDS	89
5.1. INTRODUCTION	89
5.2. MATERIALS USED IN THE EXPERIMENTAL PROGRAM	90
5.2.1. Metallic reinforcement.....	90
5.2.2. Mineral aggregates used in concrete.....	93
5.2.3. Cementitious materials.....	95
5.2.4. Water.....	97
5.2.5. Chloride concentration in the exposure solutions and grouts	98
5.3. CYCLIC POLARIZATION TESTS	99
5.3.1. Introduction and objectives.....	99
5.3.2. Experimental design and specimen layout.....	100
5.3.3. Sample preparation and test procedures	102

5.4.	GALVANIC CORROSION TESTS	103
5.4.1.	Introduction and objectives.....	103
5.4.2.	Experimental design and specimen layout – Modified ASTM G109 tests.....	103
5.4.3.	Casting, curing, and exposure procedures – Modified ASTM G109 tests	105
5.4.4.	Experimental design and specimen layout – Bearing plate tests	107
5.4.5.	Casting, curing, and exposure procedures – Bearing plate tests.....	109
5.4.6.	Corrosion evaluation procedures	110
5.5.	STRAND AND WIRE CORROSION TESTS.....	110
5.5.1.	Introduction and objectives.....	110
5.5.2.	Experimental design and specimen layout.....	111
5.5.2.1.	<i>Strand corrosion tests</i>	113
5.5.2.2.	<i>Wire corrosion tests</i>	117
5.5.3.	Concrete reaction frames and stressing operations for stressed strand specimens.....	121
5.5.3.1.	<i>Concrete reaction frames and layout</i>	121
5.5.3.2.	<i>Stressing systems</i>	123
5.5.3.3.	<i>Stressing operations</i>	126
5.5.4.	Casting, curing, and exposure procedures	128
5.5.4.1.	<i>Strand specimens</i>	128
5.5.4.2.	<i>Wire specimens</i>	129
5.5.5.	Corrosion evaluation procedures	134
5.5.6.	Tension capacity determination or estimation procedures.....	134
5.5.6.1.	<i>Strand specimens</i>	134
5.5.6.2.	<i>Wire specimens</i>	135
5.6.	SUMMARY.....	139

6. ELECTROCHEMICAL CHARACTERISTICS AND TENSION CAPACITY OF ASTM-A416 STRANDS AND WIRES.....141

6.1.	INTRODUCTION	141
6.2.	CYCLIC POLARIZATION TEST RESULTS	141
6.3.	GALVANIC CORROSION TEST RESULTS.....	144
6.3.1.	Modified ASTM G109 test results.....	144
6.3.2.	Bearing plate test results	148
6.4.	STRAND AND WIRE CORROSION TEST RESULTS	150
6.4.1.	General visual observations	150
6.4.2.	Estimating tension capacity from strand photographs.....	151
6.4.3.	Tension capacities of strand and wire specimens	153
6.4.3.1.	<i>Unstressed strand capacities under wet-dry exposure conditions</i>	153
6.4.3.2.	<i>Stressed strand capacities under wet-dry exposure conditions</i>	156
6.4.3.3.	<i>Unstressed wire capacities under wet-dry exposure conditions</i>	157

6.4.3.4.	<i>Unstressed wire capacities under continuous-atmospheric exposure conditions</i>	159
6.5.	CRITICAL PARAMETERS INFLUENCING TENSION CAPACITY OF PT STRANDS	161
6.5.1.	Statistical hypothesis tests.....	161
6.5.2.	Effect of grout class	161
6.5.3.	Effect of moisture and chloride conditions.....	162
6.5.4.	Effect of stress conditions.....	166
6.5.5.	Effect of void conditions.....	167
6.6.	SUMMARY.....	168
7.	MODELING TENSION CAPACITY OF STRANDS EXPOSED TO WET-DRY (WD) CONDITIONS	169
7.1.	INTRODUCTION	169
7.2.	STATISTICAL PROCEDURES TO DEVELOP PROBABILISTIC MODELS	169
7.2.1.	Statistical diagnosis of experimental data.....	169
7.2.2.	Formulation of probabilistic capacity models.....	169
7.2.3.	Assessment or calibration of probabilistic capacity models	171
7.2.3.1.	<i>Posterior statistics of model parameters</i>	171
7.2.3.2.	<i>Mean absolute percentage error</i>	173
7.2.3.3.	<i>Selection of the most parsimonious parameter set</i>	173
7.2.3.4.	<i>Possible high correlation coefficients between model parameters</i>	173
7.2.3.5.	<i>Validation plot</i>	174
7.3.	PROBABILISTIC MODELS FOR TENSION CAPACITY OF “AS-RECEIVED” STRANDS.....	175
7.4.	TENSION CAPACITY MODELS FOR STRANDS UNDER WD EXPOSURE CONDITIONS: ANALYTICAL PROGRAM	176
7.4.1.	Parameters used in the models for wet-dry conditions	176
7.4.2.	Analytical steps to develop capacity models for strands under wet-dry conditions (steps WD-1 through WD-5).....	177
7.4.3.	Assumptions for predicting strand capacity under wet-dry conditions	183
7.5.	TENSION CAPACITY MODELS FOR STRANDS UNDER WD EXPOSURE CONDITIONS: RESULTS.....	184
7.5.1.	Step WD-1: Identifying groups of void types with statistically dissimilar effects on tension capacity	184
7.5.2.	Step WD-2: Models for unstressed strands under WD exposure conditions (using unstressed strand data only)	184
7.5.2.1.	<i>Diagnosis of the effect of wet-dry exposure time</i>	185
7.5.2.2.	<i>Diagnosis of the effect of chloride concentration in exposure solution</i>	187

7.5.2.3.	<i>Diagnosis of the effect of interaction between chloride and time parameters</i>	190
7.5.2.4.	<i>Model formulation</i>	190
7.5.2.5.	<i>Model assessment - No Void conditions</i>	191
7.5.2.6.	<i>Model assessment - Parallel Void conditions</i>	192
7.5.2.7.	<i>Model assessment – Bleedwater, Inclined, and Orthogonal Void conditions</i>	194
7.5.2.8.	<i>Prediction of tension capacity of unstressed strands</i>	195
7.5.3.	STEP WD-3: Models for stressed strands under WD exposure conditions (using stressed strand data only).....	198
7.5.3.1.	<i>Diagnosis of the effect of wet-dry exposure time</i>	198
7.5.3.2.	<i>Diagnosis of the effect of chloride concentration in exposure solution</i>	200
7.5.3.3.	<i>Diagnosis of the effect of interaction between chloride and time parameters</i>	203
7.5.3.4.	<i>Model formulation</i>	203
7.5.3.5.	<i>Model assessment – No Void conditions</i>	204
7.5.3.6.	<i>Model assessment – Parallel Void conditions</i>	205
7.5.3.7.	<i>Model assessment – Bleedwater, Inclined, and Orthogonal Void conditions</i>	207
7.5.3.8.	<i>Prediction of tension capacity of stressed strands</i>	209
7.5.4.	STEP WD-4: Models for stressed strands under WD exposure conditions (using unstressed strand model and stressed strand data).....	211
7.5.4.1.	<i>Diagnosis of the relationship between the capacities of unstressed and stressed strands</i>	211
7.5.4.2.	<i>Model formulation</i>	213
7.5.4.3.	<i>Model assessment – No Void conditions</i>	213
7.5.4.4.	<i>Model assessment – Parallel Void conditions</i>	214
7.5.4.5.	<i>Model assessment – Bleedwater, Inclined, and Orthogonal Void conditions</i>	216
7.5.4.6.	<i>Prediction of tension capacity of stressed strands</i>	218
7.5.5.	STEP WD-5: Selection of the most appropriate set of models for WD conditions.....	219
7.5.6.	Summary of strand capacity models for wet-dry conditions	221

8. MODELING THE RELATIONSHIPS BETWEEN TENSION CAPACITIES OF STRANDS AND WIRES223

8.1.	INTRODUCTION	223
8.2.	RELATIONSHIPS BETWEEN TENSION CAPACITIES OF STRESSED STRANDS AND UNSTRESSED WIRES – ANALYTICAL PROGRAM.....	223
8.2.1.	Introduction and objectives.....	223
8.2.2.	Parameters used in the probabilistic models for tension capacity	223

8.2.3.	Analytical steps to develop “unstressed wire-stressed strand” capacity model (steps WS-1 through WS-4).....	225
8.3.	RELATIONSHIPS BETWEEN TENSION CAPACITIES OF STRESSED STRANDS AND UNSTRESSED WIRES – RESULTS	230
8.3.1.	STEP WS-1: Develop models for wires under 0.006, 0.018, and 1.8 % s_{CI} levels.....	230
	8.3.1.1. <i>Diagnosis of the effect of wet-dry exposure time</i>	230
	8.3.1.2. <i>Formulation and assessment</i>	231
8.3.2.	STEP WS-2: Develop UW-US-SS _{WD, BIOV} model.....	234
	8.3.2.1. <i>Step WS-2a: Develop Model UW-US_{WD, BIOV}</i>	234
	8.3.2.2. <i>Step WS-2b: Develop Model UW-US-SS_{WD, BIOV}</i>	237
8.3.3.	STEP WS-3: Develop UW-SS _{WD, BIOV} Model.....	239
8.3.4.	Step WS-4: Select the more suitable “wire-strand” relationship for both WD and CA exposure conditions	242
8.3.5.	Summary of wire-strand relationships.....	243

9. MODELING TENSION CAPACITY OF STRANDS EXPOSED TO CONTINUOUS-ATMOSPHERIC (CA) CONDITIONS.....245

9.1.	INTRODUCTION	245
9.2.	PROBABILISTIC MODELS FOR TENSION CAPACITY OF STRANDS – ANALYTICAL PROGRAM	245
9.2.1.	Introduction and objectives.....	245
9.2.2.	Parameters used in the probabilistic models for tension capacity	246
9.2.3.	Steps to develop tension capacity models for strands under CA conditions (steps CA-1 through CA-4).....	247
9.2.4.	Assumptions for predicting strand capacity under CA conditions	250
9.3.	TENSION CAPACITY MODELS FOR STRANDS UNDER CA EXPOSURE CONDITIONS- RESULTS	251
9.3.1.	Step CA-1: Develop probabilistic model for tension capacity of strands under CA and NV conditions.....	251
9.3.2.	Step CA-2: Develop probabilistic models for unstressed wires under CA and BIOV conditions	251
9.3.3.	Step CA-3: Predict the capacity of stressed strands at 9 months.....	256
9.3.4.	Step CA-4: Develop the model for stressed strands under CA and BIOV exposure conditions	257
9.3.5.	Prediction of tension capacity of stressed strands under CA and BIOV conditions.....	258

9.4. SUMMARY.....	259
-------------------	-----

10. MODELING AND ASSESSMENT OF STRUCTURAL RELIABILITY OF SEGMENTAL BRIDGES.....261

10.1. INTRODUCTION AND OBJECTIVE	261
10.2. A FRAMEWORK TO DETERMINE STRUCTURAL RELIABILITY	261
10.3. MODELING STRENGTH RELIABILITY	263
10.3.1. Strength limit state function, probability of strength failure, and strength reliability index	263
10.3.2. Probabilistic modeling of moment capacity of the girder at mid-span	264
10.3.2.1. <i>The main function to determine moment capacity</i>	266
10.3.2.2. <i>The subfunction to determine bending moment corresponding to a curvature</i>	269
10.3.2.3. <i>The subfunction to determine the total normal tensile forces</i>	272
10.3.3. Probabilistic modeling of moment demand on the girder at midspan	274
10.4. MODELING SERVICE RELIABILITY	274
10.4.1. Service limit state function, probability of service failure, and service reliability index	274
10.4.2. Probabilistic modeling of stress capacity of extreme fibers at midspan	276
10.4.3. Probabilistic modeling of stress demand on extreme fibers at mid-span.....	276
10.5. RANDOM PARAMETERS IN THE PROBABILISTIC MODELS	278
10.5.1. Void and damage/opening conditions on post-tensioning systems	278
10.5.2. Tension capacity of strands.....	279
10.5.3. Prestress loss of strands	281
10.5.4. Compressive strength of concrete.....	282
10.5.5. Dead and live load parameters.....	282
10.6. TIME-VARIANT STRUCTURAL RELIABILITY ASSESSMENT OF TYPICAL POST-TENSIONED BRIDGE	283
10.6.1. Geometrical and structural characteristics of a typical post-tensioned bridge	283
10.6.2. Definitions of parameters for the reliability assessment.....	285
10.6.3. Time-variant strength reliability index	288
10.6.3.1. <i>Strength reliability when exposed to 0.006 %sCI⁻ conditions</i>	289
10.6.3.2. <i>Strength reliability when exposed to 0.018 %sCI⁻ conditions</i>	290
10.6.3.3. <i>Strength reliability when exposed to 1.8 %sCI⁻ conditions</i>	291
10.6.4. Time-variant service reliability index	292
10.6.4.1. <i>Service reliability when exposed to 0.006 %sCI⁻ conditions</i>	293
10.6.4.2. <i>Service reliability when exposed to 0.018 %sCI⁻ conditions</i>	294
10.6.4.3. <i>Strength reliability when exposed to 1.8 %sCI⁻ conditions</i>	295

10.7. SUMMARY.....	296
11. CONCLUSIONS AND RESEARCH RECOMMENDATIONS	297
11.1. INTRODUCTION	297
11.2. LIMITATIONS AND ASSUMPTIONS.....	297
11.3. CONCLUSIONS.....	298
11.3.1. Corrosion Risks at Different Geographic Locations in Texas.....	298
11.3.2. Cyclic Polarization Curves of ASTM A416 Steel	298
11.3.3. Galvanic Corrosion Characteristics of PT Systems.....	299
11.3.4. Probabilistic Tension Capacity of Strands and Wires.....	299
11.3.5. Structural reliability of post-tensioned bridges.....	301
11.4. RECOMMENDATIONS FOR FUTURE RESEARCH	302
11.5. RECOMMENDATIONS FOR FIELD IMPLEMENTATIONS	304
REFERENCES.....	305
APPENDIX A. ALL TENSION CAPACITY DATA	315
APPENDIX B. STRUCTURAL DETAILING OF CONCRETE REACTION	
FRAMES.....	333
APPENDIX C. RESULTS FROM RELIABILITY ASSESSMENT	337

LIST OF FIGURES

Figure 1-1. Cross-Section of a PT Box Girder with Internal and External Tendons.....	5
Figure 1-2. Typical Cross-Sectional Views of Tendons with and without Voids.	8
Figure 1-3. Schematic Showing Various Elements of Experimental-Analytical Program.....	11
Figure 2-1. Collapsed Ynys-y-Gwas Bridge (YouTube 2009).....	16
Figure 2-2. Niles Channel Bridge, Florida Keys, Florida (Structurae 2009).....	18
Figure 2-3. Mid-Bay Bridge, Choctawhatchee Bay, Florida (Worldbreak 2009).	18
Figure 2-4. (a) Tendon Corrosion near Anchorage Zone, (b) Tendon Corrosion at Anchorage, (c) An Exposed Strand along a Bleed Water Trail, and (d) Cracked PT Ducts in the Mid-Bay Bridge, Florida (FDOT 2001a).	19
Figure 2-5. Bob Graham Sunshine Skyway Bridge, Tampa Bay, Florida (Wikipedia 2009).	20
Figure 2-6. Severely Corroded Strands at Anchorage Zones of PT Columns in the Bob Graham Sunshine Skyway Bridge, Tampa Bay, Florida (FDOT 2001b).	21
Figure 2-7. Varina-Enon Bridge, over the James River, Virginia (Roadstothe future 2009).	22
Figure 2-8. Tendon Corrosion in Varina-Enon Bridge, Virginia (Hansen 2007).	22
Figure 2-9. Pitting Corrosion on ASTM A 53 Pipe and ASTM A 416/A 416M-99 Strand Due to Coupled Galvanic- and Chloride-Induced Corrosion (FDOT 2002).	23
Figure 2-10. Elevations Showing Typical Void Locations in Grouted Tendon Systems.	26
Figure 2-11. Variation of Pore Solution Composition with Time of Hydration (Diamond 1981).	28
Figure 2-12. Carbonated GAS Interfaces in Anchorage Zones.	29
Figure 2-13. Daily Precipitation in San Antonio, Texas (NCDC 2009).	31
Figure 2-14. Thickness Loss of Steel Surfaces on Ships Exposed to Seawater (Predicted Using Existing Corrosion Models).	37
Figure 2-15. Thickness Loss of Carbon Steel Due to Atmospheric Corrosion (Predicted Using Existing Power Models) (Note: 1 inch = 25.4 mm).	43
Figure 2-16. HL-93 Loading Specified by AASHTO (1998, 2007): (a) Truck and Uniform Lane Load and (b) Tandem and Uniform Lane Load.	56

Figure 2-17. Locations of Existing and Future Post-Tensioned, Segmental Concrete Bridges in Texas.	64
Figure 2-18. A Layout and Phases of San Antonio “Y” Bridge (Wollmann et al. 2001).....	65
Figure 2-19. Estimated Trucks Carrying US-Mexico Trade on US Highway Corridors to and from the Lower Rio-Grande Valley Region (McGray 1998).....	66
Figure 2-20. Estimated Trucks Carrying US-Mexico Trade on US Highway Corridors to and from the Laredo Region (McGray 1998).	66
Figure 2-21. Typical Cross-Sections of Four Types of Girders in San Antonio “Y” Bridge.	68
Figure 2-22. Symbolic Notations for Cross-Sections of Segmental Box Girders.	72
Figure 4-1. Mean Daily Maximum Temperature in the United States.	80
Figure 4-2. Number of Freeze Days in Texas (Data Collected from 1995-2000).	81
Figure 4-3. Annual Average Temperature in Texas (Data Collected from 1961-1990).....	82
Figure 4-4. Annual Average Relative Humidity in Texas (Data Collected from 1961-1990).	82
Figure 4-5. Number of Rain-Days in Texas (Data Collected from 1961-1990).	83
Figure 4-6. Total Chloride Factor.	85
Figure 4-7. Quantitative Assessment of Total Corrosion Risk.	86
Figure 4-8. Qualitative Corrosion Risk Level.....	87
Figure 5-1. Experimental Program (Electrochemical and Tension Capacity Behavior of Wires and Strands).....	89
Figure 5-2. Observed Tension Capacities of “As-Received” Strands and Wires.	93
Figure 5-3. Particle Size Distribution Curves for Fine and Aggregates.	94
Figure 5-4. Schematic of the Cyclic Polarization Test Setup (Note: Not Drawn to Scale).....	101
Figure 5-5. Cyclic Polarization Test Setup (Left: Corrosion Cell [Perkin Elmer, Inc.], Right: Potentiostat [Solartron Model 1287], and a Laptop Computer).	101
Figure 5-6. Photograph and Schematic of a Modified ASTM G109 Test Specimen.	105
Figure 5-7. Photographs and Schematics of a Bearing Plate Test Specimen.	108
Figure 5-8. Experimental Designs for All the Strand and Wire Corrosion Tests.	112

Figure 5-9. Schematics of Strand Corrosion Test Specimens.....	116
Figure 5-10. Schematic and Photograph of Wire Corrosion Test Specimen.....	120
Figure 5-11. Schematic and Photograph of Vertical Concrete Reaction Frame with Stressed Strand Specimens.....	122
Figure 5-12. Schematic and Photograph of Horizontal Concrete Reaction Frames with Stressed Strand Specimens.	123
Figure 5-13. Schematics and Photographs of (a) Live End and (b) Dead End of the Stressed Strand on a Concrete Reaction Frame.	124
Figure 5-14. LVDT Setup to Measure the Strand Shortening during Stressing Operation.	125
Figure 5-15. Schematic and Photograph of Strand Stressing Setup.	127
Figure 5-16. Exposure of Wire Specimens to WD Conditions.....	130
Figure 5-17. Exposure of Wire Specimens to CA Conditions with 100 %RH.....	131
Figure 5-18. Exposure of Wire Specimens to CA Conditions with 45 and 70 %RH.	133
Figure 5-19. Cross-Sectional Area Measurement Using Surface Replicating Media and Profilometer.	137
Figure 5-20. A Graphical Representation of the Analytical Determination of Tension Capacity of Wire Specimens under 9-Month CA Exposure.....	138
Figure 6-1. Cyclic Polarization Curves for ASTM A416 Steel.....	142
Figure 6-2. Total Corrosion on the Modified ASTM G109 Samples with Wet-Dry Exposure to 0 %sCl ⁻ Solution.....	146
Figure 6-3. Total Corrosion on the Modified ASTM G109 Samples with Wet-Dry Exposure to 9 %sCl ⁻ Solution.....	147
Figure 6-4. Total Corrosion on the Bearing Plate Samples (0 %sCl ⁻ Solution).....	149
Figure 6-5. Total Corrosion on the Bearing Plate Samples (9 %sCl ⁻ Solution).....	149
Figure 6-6. Micrographs Showing (a) Localized Corrosion near Grout-air-strand (GAS) Interface; (b) Linear and Pitting Patterns of Corrosion.	151
Figure 6-7. Cleaned Strand Surface, Corrosion Characteristics, and Residual Tension Capacity.	152
Figure 6-8. Tension Capacities of Unstressed Strands Exposed to NV and PV Conditions.	154

Figure 6-9. Tension Capacities of Unstressed Strands Exposed to OV, IV, BV, and BIOV Conditions.....	155
Figure 6-10. Tension Capacities of Stressed Strands Exposed to NV, PV, and BIOV Conditions.....	157
Figure 6-11. Tension Capacities of Unstressed Wires Exposed to WD and OV Conditions.	158
Figure 6-12. Tension Capacities of Unstressed Wires Exposed to CA and OV Conditions.	160
Figure 6-13. Tension Capacity of Unstressed and Stressed Strand Specimens at the End of 12-Months of WD Exposure.....	164
Figure 7-1. Flowchart for Developing the Tension Capacity Models for Strands in WD Conditions.	179
Figure 7-2. Graphical Representations of Analytical Steps WD-2, WD-3, and WD-4.....	182
Figure 7-3. Annual Wet-Time for Tendons (Based on Assumed Values of Time Required to Dry the Tendons).	183
Figure 7-4. Effect of Wet-Dry Exposure Time on Capacity of Unstressed Strands (with $\phi_{wet} = 0.5$).....	186
Figure 7-5. Effect of %sCl ⁻ on Capacity of Unstressed Strands (with $\phi_{wet} = 0.5$).....	188
Figure 7-6. Effect of ln[%sCl ⁻] on Capacity of Unstressed Strands (with $\phi_{wet} = 0.5$).....	189
Figure 7-7. Validation Plot for the Model US _{WD, NV}	192
Figure 7-8. Validation Plot for the Model US _{WD, PV}	194
Figure 7-9. Validation Plot for the Model US _{WD, BIOV}	195
Figure 7-10. Capacity of Unstressed Strands under WD Exposure to Various Chloride Solutions (Predicted Using US _{WD} Models).	197
Figure 7-11. Effect of Wet-Dry Exposure Time on Capacity of Stressed Strands (with $\phi_{wet} = 0.5$).....	199
Figure 7-12. Effect of %sCl ⁻ on Capacity of Stressed Strands (with $\phi_{wet} = 0.5$).....	201
Figure 7-13. Effect of ln[%sCl ⁻] on Capacity of Stressed Strands (with $\phi_{wet} = 0.5$).....	202
Figure 7-14. Validation Plot for the Model SS _{WD, NV}	205
Figure 7-15. Validation Plot for the Model SS _{WD, PV}	207

Figure 7-16. Validation Plot for the Model $SS_{WD, BIOV}$	209
Figure 7-17. Capacity of Stressed Strands under WD Exposure ($\phi_{wet} = 0.17$) to Various Chloride Solutions (Predicted Using SS_{WD} Models).....	210
Figure 7-18. Scatter Plots between the Capacities of Unstressed and Stressed Strands.....	212
Figure 7-19. Validation Plot for US- $SS_{WD, NV}$ Model.....	214
Figure 7-20. Validation Plot for US- $SS_{WD, PV}$ Model.....	216
Figure 7-21. Validation Plots for US- $SS_{WD, BIOV}$ Model.....	217
Figure 7-22. Capacity of Stressed Strands under WD Exposure ($\phi_{wet} = 0.17$) to Various Chloride Solutions (Predicted Using US- SS_{WD} Models).....	219
Figure 8-1. Flowchart for Developing the “Wire-Strand” Capacity Relationships.....	226
Figure 8-2. Graphical Representation of Step WS-1.....	228
Figure 8-3. Graphical Representations of Steps WS-2a and WS-3.....	229
Figure 8-4. Scatter Plot between Capacity of Unstressed Wires, $C_{T,UW}$, and Wet-Dry Exposure Time, t_{WD}	231
Figure 8-5. Validation Plot for the $UW_{WD, 0.006}$ Model.....	233
Figure 8-6. Validation Plot for the $UW_{WD, 0.018}$ Model.....	233
Figure 8-7. Validation Plot for the $UW_{WD, 1.8}$ Model.....	234
Figure 8-8. Scatter Plot between Predicted Capacity of Unstressed Wires, $C_{T,UW}$, and Observed Capacity of Unstressed Strands, $C_{T,US}$	235
Figure 8-9. Validation Plots for $UW-US_{WD}$ Model.....	237
Figure 8-10. Validation Plot for $UW-US-SS_{WD, BIOV}$ Model.....	239
Figure 8-11. Scatter Plot between Predicted Capacity of Unstressed Wires, $C_{T,UW}$, and Observed Capacity of Stressed Strands, $C_{T,SS}$	240
Figure 8-12. Validation Plot for the $UW-SS_{WD, BIOV}$ Model.....	242
Figure 9-1. Flowchart for Developing the Tension Capacity Models for Strands under CA and BIOV Exposure Conditions.....	249
Figure 9-2. Graphical Representations of Steps CA-2, CA-3, and CA-4.....	250
Figure 9-3. Scatter Plot between % gCl^- and Wire Capacity at 9-Month CA Exposure.....	252

Figure 9-4. Scatter Plot between %RH and Wire Capacity at 9-Month CA Exposure.....	252
Figure 9-5. Scatter Plot between T and Wire Capacity at 9-Month CA Exposure.....	253
Figure 9-6. Scatter Plot between the 3-Way Interaction Term and Wire Capacity at 9-Month CA Exposure.....	254
Figure 9-7. Validation Plot for the $UW_{CA, BIOV}$ Model.....	256
Figure 9-8. Capacity of Stressed Strands under CA Conditions with $n_{CA} = -0.005$	259
Figure 10-1. A Framework to Model and Assess the Generalized Reliability Index, β	262
Figure 10-2. Strain, Stress, and Normal Force Distributions on a T-girder.....	265
Figure 10-3. Simplified Flowchart to Determine Moment Capacity of a PT Girder.....	268
Figure 10-4. Schematic Showing the Movement of Curvature, ϕ	269
Figure 10-5. Simplified Flowchart to Determine Moment Corresponding to a Curvature.....	271
Figure 10-6. Simplified Flowchart to Determine the Total Normal Tensile Force, F_T	273
Figure 10-7. Schematic Showing the Movement of Curvature, ϕ	278
Figure 10-8. Strategy to Select the Strand Capacity Models Based on Exposure Conditions.....	281
Figure 10-9. Cross-Section at Mid-span of the Typical PT Girder Used in This Study.....	284
Figure 10-10. Parameter Combinations for the Reliability Assessment Program.....	286
Figure 10-11. Strength Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (0.006 %sCI).....	289
Figure 10-12. Strength Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (0.018 %sCI).....	290
Figure 10-13. Strength Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (1.8 %sCI).....	291
Figure 10-14. Service Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (0.006 %sCI).....	293
Figure 10-15. Service Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (0.018 %sCI).....	294
Figure 10-16. Service Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (1.8 %sCI).....	295

LIST OF TABLES

Table 2-1. Percentage of Voided Tendons among the External Tendons on San Antonio “Y” Bridge (Adapted from TxDOT [2004]).	27
Table 2-2. Existing Models for Thickness Loss of Steel Surfaces on Ships Exposed to Seawater.	37
Table 2-3. Existing Atmospheric Corrosion Models (as a Function of Environmental Parameters and Exposure Time) for Steel.	39
Table 2-4. Existing Power Models for Atmospheric Corrosion of Carbon Steel.	42
Table 2-5. Statistics of Concrete Compressive Strength (Nowak and Collins 2000).	49
Table 2-6. Compressive and Tensile Stress Limits for Fully Prestressed Components in Segmental Concrete at Service Limit State after Losses (AASHTO 2007).	53
Table 2-7. Dead Load Parameters from TxDOT Bridge Drawings.	55
Table 2-8. Governing Limit States for Pre-tensioned Girders with Short and Long Spans.	58
Table 2-9. Summary of β_{target} and P_f (Adapted from ISO 13822).	62
Table 2-10. Span and Girder Inventory of San Antonio “Y” Bridge.	69
Table 2-11. Quantity and Location of Longitudinal Tendons at Midspan on San Antonio “Y” Bridge.	71
Table 2-12. Typical Geometrical Parameters of Box Girders in San Antonio “Y” Bridge.	72
Table 4-1. Chloride Usage Factor.	84
Table 4-2. Chloride Frequency Factor.	84
Table 4-3. Total Corrosion Risk Factors.	86
Table 5-1. Different Steel Reinforcement Types Used in this Research Program.	90
Table 5-2. Representative Chemical Compositions of Steel Reinforcement Types Used.	91
Table 5-3. Material Characteristics of Fine and Coarse Aggregates.	95
Table 5-4. Representative Chemical Compositions of Cementitious Materials Used.	96
Table 5-5. Representative Mixture Proportions of Concrete and Grout Used.	96
Table 5-6. Concentrations of Impurities in the Water Used in this Research Program.	98

Table 5-7. Chemical Compositions of Exposure Solutions Used in this Research.	99
Table 5-8. Experimental Design for Modified ASTM G109 Test Program.	104
Table 5-9. Experimental Design for Bearing Plate Test Program.	108
Table 5-10. Experimental Design Showing the Number of Test Specimens in the Unstressed Strand Corrosion Test under WD Exposure Conditions*	113
Table 5-11. Experimental Design Showing the Number of Test Specimens in the Stressed Strand Corrosion Test under WD Exposure Conditions*	114
Table 5-12. Experimental Design Showing the Number of Test Specimens in the Unstressed Wire Corrosion Test under WD Exposure Conditions*	118
Table 5-13. Experimental Design Showing the Number of Test Specimens in the Unstressed Wire Corrosion Test under CA Exposure Conditions*	119
Table 6-1. Approximate Active, Passive, and Transpassive Potential Regions for the ASTM A416 Steel.....	144
Table 6-2. Mean [‡] , Coefficient of Variation (COV), and Mean Capacity Loss ($C_{T,loss}$)* of Tension Capacities Observed at the End of 12-Month WD Exposure.	165
Table 6-3. Groups of Void Conditions with Similar Effects on Tension Capacity Loss.....	168
Table 7-1. List of Tension Capacity Models for Strands under WD Exposure Conditions.	178
Table 7-2. MAPE and Posterior Statistics of the $US_{WD, NV}$ Model.	191
Table 7-3. MAPE and Posterior Statistics of the $US_{WD, PV}$ Model.	193
Table 7-4. MAPE and Posterior Statistics of the $US_{WD, BIOV}$ Model.	194
Table 7-5. MAPE and Posterior Statistics of the $SS_{WD, NV}$ Model.....	204
Table 7-6. MAPE and Posterior Statistics of the $SS_{WD, PV}$ Model.	206
Table 7-7. MAPE and Posterior Statistics of the $SS_{WD, BIOV}$ Model.	208
Table 7-8. MAPE and Posterior Statistics of the $US-SS_{WD, NV}$ Model.	214
Table 7-9. MAPE and Posterior Statistics of the $US-SS_{WD, PV}$ Model.....	215
Table 7-10. MAPE and Posterior Statistics of the $US-SS_{WD, BIOV}$ Model.	216
Table 7-11. MAPEs of US_{WD} , SS_{WD} , and $US-SS_{WD}$ Models.....	220
Table 7-12. Model Error Coefficients (σ) of US_{WD} , SS_{WD} , and $US-SS_{WD}$ Models.....	220

Table 8-1. List of Wire and “Wire-Strand” Capacity Models for BIOV Conditions.	225
Table 8-2. MAPE and Posterior Statistics of UW_{WD} Models.	232
Table 8-3. MAPE and Posterior Statistics of $UW-US_{WD, BIOV}$ Full Model.	236
Table 8-4. MAPE and Posterior Statistics of Full $UW-SS_{WD, BIOV}$ Model.	241
Table 9-1. MAPE and Posterior Statistics of $UW_{CA, BIOV}$ Model.	254
Table 10-1. Definitions of the Terms in the Strand Capacity Models.	280
Table 10-2. Posterior Statistics of Model Parameters in the Strand Capacity Models.	280
Table 10-3. Effective Flange Widths for Each Side of the Web.	284
Table 10-4. Average Annual Temperature and Relative Humidity Conditions in Some Major Cities in the US (Source: www.cityrating.com).	287
Table 10-5. Random Parameters for Reliability Assessment*.	287

EXECUTIVE SUMMARY

Post-tensioned (PT) bridges are effective structures from both engineering and economical perspectives. These bridge types have the ability to span long lengths and are often unique, graceful, and the most economical alternative. During construction, PT tendons are used to hold the precast girders together, resulting in a composite structure. The serviceability and performance of PT bridges are very much dependent on the tendons—without the tendons PT bridges would collapse. In the late 1990s and early 2000s, state highway agencies (SHAs) identified voids in the grouted tendons. At some of these void locations, the strands in the tendons were exhibiting corrosion deterioration—some significant. The Texas Department of Transportation (TxDOT) inspected several of its PT bridges and also found voids in the tendons. A consultant’s report recommended that these voids be filled. However, these bridges are exposed to very different environmental conditions than the bridges that exhibited severe corrosion of the strands, so the question was raised regarding the necessity to repair the voids and thus resulted in this research project: TxDOT 0-4588, “Effect of Voids in Grouted, Post-Tensioned Concrete Bridge Construction.”

The objectives of TxDOT 0-4588 were to provide guidance on the corrosivity of PT strands under different environmental conditions, to generate a general model to assess the reliability of PT bridges exhibiting corrosion of the strands, to provide recommendations for repair grout materials, and to provide recommendations for the inspection and repair of PT systems.

In late 2007, a PT bridge in Virginia experienced failure of a tendon. The tendon had been recently repaired (i.e., grouted), and it was believed that the failure could be a result of galvanic cells formed by the different grout environments. The State of Virginia placed a moratorium on repairing PT bridges, and TxDOT delayed repairs pending the findings from Virginia Department of Transportation. Unfortunately, the potential corrosion of strands resulting from a galvanic couple caused by embedment in different grout materials was not investigated as part of this research program and no recommendations can be made on this mechanism of deterioration.

The research did perform extensive studies on the corrosion of strands in various environments. Results indicate that significant decreases in strand capacity occur when the strands are exposed directly to moisture and moisture containing chlorides. The research indicated that high humidity levels do not result in the same deterioration as the water and chloride exposure conditions; however, condensation can lead to higher corrosion rates. The results also indicated that oxygen and carbon dioxide levels have limited influence on the reduction of strand capacities. Using this information, the researchers developed a strand capacity model for the different environmental exposure conditions. Using a general cross section of a PT bridge, the researchers performed reliability analyses. These analyses indicated that a PT bridge could fail as soon as 21 years after construction if the strands are subjected to high chloride containing solutions. If the tendons are protected from these aggressive environments, the service life of these structures, based on corrosion of the strands, could exceed 100 years. The research findings show the importance of protecting the strands from water and chlorides.

To this end, the researchers evaluated different inspection and repair techniques for tendons on PT bridges. The researchers also assessed the applicability of the current TxDOT specification for repair grouts. The research found that the pressure-vacuum repair procedure results in the best fillability of the voids. This method was also identified as being the fastest and most economical. The research also proposed a new method for assessing repair grouts. These repair methods and materials can be used if it is determined that galvanic cells do not form between the existing and repair grouts.

The researchers recommend that every effort is made to keep water and chlorides from the inside of PT girders. This should include the repair of drainage pipes, damaged ducts, or leaking anchorage plates, or correcting any other issues that result in water penetrating the tendon and girder. Preventing the infiltration of water and chlorides into the tendons by performing these relatively simple tasks will likely extend the service life of PT bridges in Texas. The researchers also recommend that a study be performed on the influence of different grout types on corrosion of strands.

1. INTRODUCTION

Equation Section (Next)

1.1. PRESTRESSED CONCRETE TECHNOLOGY

In the late 1920s, Eugene Freyssinet, a French civil engineer, pioneered the prestressed concrete technology. He patented prestressed concrete technology in 1928 and is considered the father of prestressed concrete (Emmanuel 1980). Although Freyssinet pioneered prestressed concrete, Doehring patented prestressing methods as early as 1888. Freyssinet recognized that only high-strength prestressing wire could counteract the effects of creep, develop anchorage, and improve other load-carrying attributes, which helped in the widespread use of prestressed concrete technology in many structural systems, including long-span segmental bridges. Two types of stressing technologies are commonly used. These include 1) pre-tensioning, where the stress is applied before the concrete hardens; and 2) post-tensioning, where the stress is applied after the concrete hardens. This document focuses on the electrochemical characterization and probabilistic capacity modeling of post-tensioning strands and structural reliability of grouted, post-tensioned (PT), segmental concrete bridges (denoted as “PT bridges” herein).

1.2. GROUTED, POST-TENSIONED, SEGMENTAL CONCRETE BRIDGES

In the 1950s, Europeans started the construction of long-span PT bridges. About a decade later, the United States (US) also began constructing similar PT bridges. Later, grouted post-tensioned systems became economically viable and popular for long-span PT bridge construction (NCHRP 1998). The definitions of some important terminologies used in this document are provided next.

1.3. DEFINITIONS

Various components of grouted PT systems include wires, strands, ducts, and tendons. In this document, they are defined as follows:

- Wire (or PT Wires) – Single wire with 0.2-inch (5 mm) diameter, made of high-strength steel meeting ASTM A416 specifications.

- Strand (or PT Strands) – Seven helically coiled wires (six outer wires helically coiled around one center wire) with a nominal diameter of 0.6 inches (15.24 mm).
- Ducts (or PT Ducts) – Metallic or high-density polyethylene (HDPE) pipe in which several strands are placed and then the interstitial spaces are filled with cementitious grouts.
- Tendons (or PT Tendons) – The system containing a group of several strands (structural load-carrying elements) and the cementitious grout and ducts (non-structural, corrosion-protection elements).
- Grout – The cementitious grout placed around the strands and inside the ducts in a tendon system.
- Void – The air space inside a PT duct system formed due to the absence of grout.

1.4. CLASSIFICATION OF POST-TENSIONED SYSTEMS

Based on the location of the tendons, grouted PT systems are classified into two types, namely internal and external PT systems. A tendon that is placed outside the concrete is defined as an external tendon. A tendon that is placed inside the concrete is defined as an internal tendon. In general, segmental, PT bridges may have either or both of these tendon systems. Figure 1-1 shows a schematic of a cross-section at mid-span of a typical PT bridge girder. In this figure, the T1 through T3 tendons are external and the T4 through T9 tendons are internal.

Tendons are also classified as bonded and unbonded tendons. A tendon that is in direct contact or bonded to the adjacent concrete is defined as a bonded tendon. A tendon that is not in direct contact with concrete or cannot transfer the stress through the surface bonding is defined as an unbonded tendon. In general, external tendons are considered unbonded tendons and internal tendons are considered bonded tendons when they are completely filled with grout and have no voids. Voids, if present, can cause discontinuity in stress transfer to the adjacent concrete along the tendon length. Hence, in this document, internal tendons are considered unbonded tendons. Following is a discussion on internal and external PT systems.

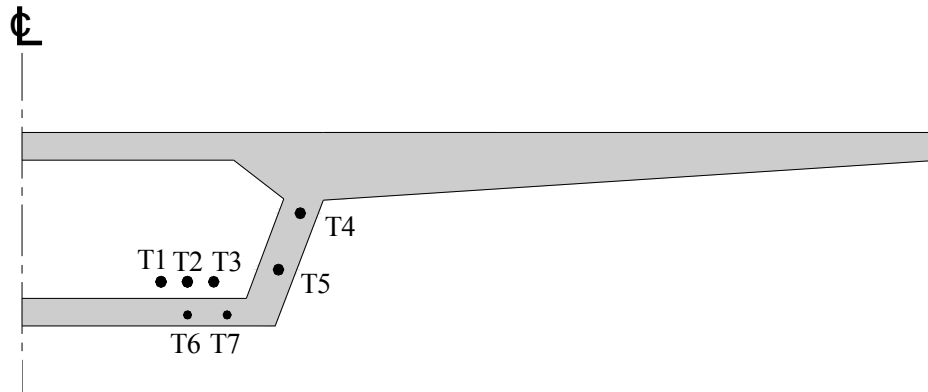


Figure 1-1. Cross-Section of a PT Box Girder with Internal and External Tendons.

1.4.1. Internal post-tensioned systems

In an internal PT system, the tendons are located or embedded inside the reinforced concrete box section. In other words, the steel strands are placed inside metallic or HDPE ducts that are embedded inside the hardened concrete. Also, the interstitial spaces between the strands and ducts are supposed to be filled with cementitious grout. Although the grout, duct, and concrete components assist in protecting the strands from external corrosive environments, corrosion of the internal PT system resulted in the sudden collapse of the Bickton Meadows footbridge in 1967 and the Ynys-y-Gwas Bridge in 1985 (NCHRP 1998). These sudden bridge collapses played a major role in eliciting a moratorium in 1992 that banned the construction of new, bonded, grouted PT bridges in the United Kingdom (UK). In 1996, the moratorium on grouted PT, cast-in-place bridge construction in the UK was removed. However, because of concerns with the corrosion protection of internal tendons at the joints between the precast segments, the moratorium on grouted PT, precast, segmental bridge construction in the UK remains in place even today.

In the recently constructed bridges in the US, this potential problem of internal tendon corrosion at box-girder or segment joints has been minimized by replacing the older practice of constructing with dry-joints with epoxy resin-joints. Contrary to the experience in the UK, the internal PT systems in US bridges have been reported as performing well (NCHRP 1998). Based on the tendon failure cases in US bridges, the internal PT system seems to be less vulnerable to corrosion than the external PT strands.

1.4.2. External post-tensioned systems

In an external PT system, the tendons are located inside the interior void space (typically rectangular or trapezoidal in cross-section) of the concrete box girder and not embedded in the hardened concrete. The external tendons are connected to the concrete box at anchorage zones and deviator blocks. The deviator blocks are used only to control tendon profile. The steel strands are placed inside HDPE ducts, and the interstitial space between the strands and the HDPE ducts is filled with cementitious grout. Because the tendons are not embedded inside the hardened concrete section, the monitoring, repair, and maintenance of external PT systems are not as complex as those for internal PT systems. However, because of the absence of concrete cover protection and the possible presence of unwanted air-voids, external tendons can be more vulnerable to corrosion than internal tendons within the same bridge segment. Tendon failures have been reported on the Mid-Bay, Niles Channel, Sunshine Skyway, and 17 other PT bridges in Florida (FDOT 1999, FDOT 2001a, FDOT 2001b, NCHRP 1998) and the Varina–Enon PT bridge in Virginia (Hansen 2007). The literature cites the presence of voids and exposure to corrosive environments as major causes for these tendon failures. These external PT system failures were observed in bridges at relatively young ages (i.e., between 8 and 17 years after construction).

1.5. RESEARCH MOTIVATION

Although grouted PT systems gained acceptance and popularity due to good economy, better aesthetics, faster construction, and other positive aspects, the PT, segmental bridge industry witnessed corrosion-related failures of grouted PT systems at relatively young ages. This raises questions on the long-term performance of these infrastructure systems. According to NCHRP (1998), “there is a pressing need for US bridge engineers to gain an understanding of durability issues associated with segmental construction and to be able to judge on a technical and rational basis the veracity of the on-going moratorium in the UK pertaining to segmental construction...” Moreover, various studies on the tendon failure cases and recent inspections conducted by various federal and state transportation agencies reported the presence of air-voids (voids herein) in the grouted tendons as one of the causes for strand corrosion (ASBI 2000, FDOT 1999, FDOT 2001a, FDOT 2001b, Hansen 2007, NCHRP 1998).

Figure 1-2 shows cross-sectional views of tendons with and without voids. Bleed-water evaporation, poor grouting, poor construction practices, or a combination of these are possible reasons for this unwanted void formation inside the tendons (FDOT 1999, FDOT 2001a, FDOT 2001b, NCHRP 1998, Schupack 2004). The strands in the voids with corrosive conditions, such as rainwater, seawater, salt-fog, de-icing/anti-icing salts, or a combination of these, can result in a higher probability of corrosion, especially localized corrosion, resulting in a reduced tension capacity. The reduced tension capacity of these strands can adversely affect the structural capacity and reliability of PT bridges. According to Poston et al. (2003), “depending upon the initial prestress in the tendon, a reduction in strength to 75 percent of the original minimum specified reduces the live-load capacity by 50 percent or more....” These studies and field observations indicate that there is a dire need to assess the long-term corrosion and structural reliability of PT bridges with voided tendons. These assessments will assist engineers in strategizing and developing better repair and maintenance programs to mitigate the risks and to ensure safe, reliable, and long-lasting PT bridges.

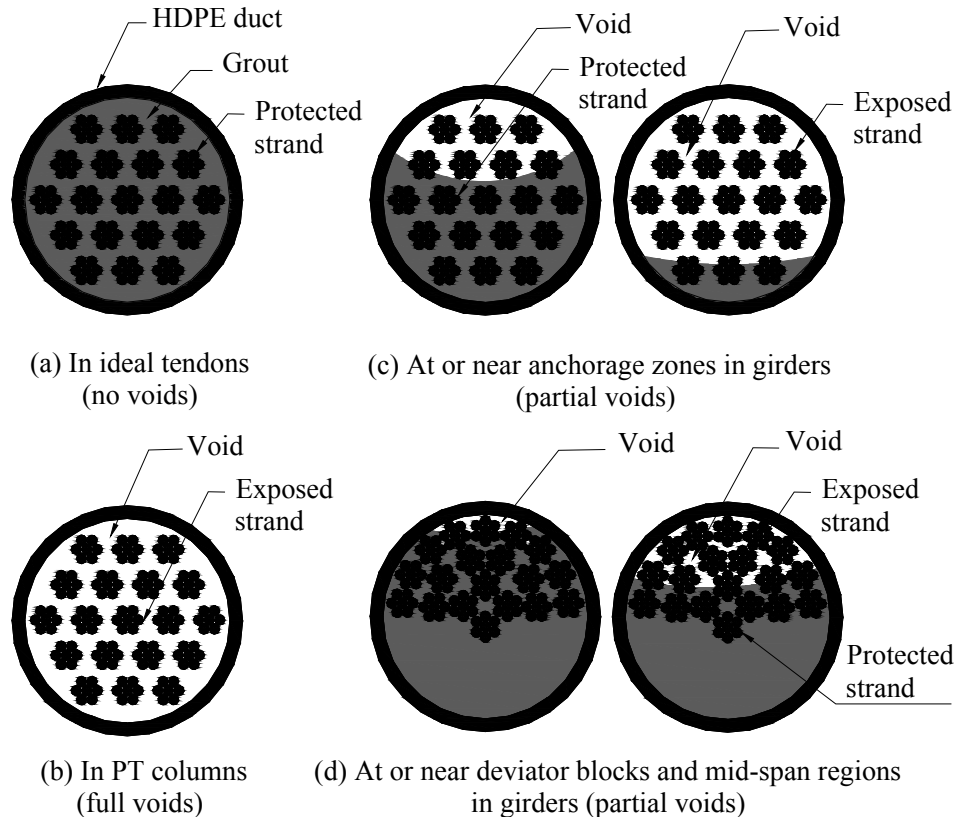


Figure 1-2. Typical Cross-Sectional Views of Tendons with and without Voids.

1.6. RESEARCH OBJECTIVES

The major objectives of this research are:

1. to develop environmental characterization maps of Texas to assist in assessing the corrosion risk level of a PT bridge based on its geographic location (results presented in Section 4);
2. to investigate the electrochemical corrosion characteristics of the steel meeting the ASTM A416 specifications when exposed to cementitious material environments with and without chloride contamination (results presented in Section 6);
3. to test whether galvanic corrosion occurs between the conventional reinforcement and strands and bearing plates at the anchorage zones, and if so, to assess the increased level of corrosion activity (results presented in Section 6);

4. to identify critical material, environmental, void, and stress parameters that influence corrosion and tension capacity of strands in PT bridges (results presented in Section 6);
5. to develop probabilistic capacity models for the strands exposed to various material, environmental, void, and stress conditions (results presented in Sections 7, 8, and 9); and
6. to develop a time-variant structural reliability model for typical PT bridges and then assess the structural reliability of a typical PT bridge (results presented in Section 10).

1.7. RESEARCH ASSUMPTIONS

To attain the research objectives, the following research hypotheses and assumptions have been made:

- Exposure to high moisture, temperature, and chloride conditions reduces the capacity of PT wires and strands. The capacity is further reduced as a function of time.
- Environmental conditions at the strand interface can be determined.
- The reduced capacity of corroding PT wires and strands can be probabilistically modeled with reasonable levels of accuracy.
- No theoretical model could be developed or found in the literature for the time-dependent corrosion of strands exposed to wet-dry (WD) conditions, continuous-atmospheric (CA) conditions, or both of these conditions. Hence, all the models developed in this document are empirical in nature.
- There exists a relationship between wire and strand corrosion processes, when exposed to similar WD or CA conditions.
- The long-term corrosion of strands under field climatic conditions can be modeled using the experimental data from shorter-term wire corrosion tests under laboratory conditions representing field climatic conditions.
- The stressed strands (i.e., strands experiencing in-service stress conditions) may exhibit more reduction in capacity than the unstressed strands.
- Because most tendons in the field were found to have voids, all the internal and external tendons are assumed to be unbonded tendons.

- Only external tendons are assumed to exhibit corrosion-induced capacity loss. Internal tendons are relatively well protected from the external environment and are assumed to be free of active corrosion.
- Among the external tendons, the rate of corrosion of strands exposed to no void (NV) and CA conditions is negligible. In addition, it is highly unlikely that there will be a parallel void (PV) and not a bleedwater, inclined, or orthogonal void (BIOV) condition (see detail definitions and schematics in Subsection 5.5.2.1) inside a tendon.
- Moment capacity and stresses at extreme fibers at midspan of a PT bridge girder can be modeled using probabilistic models for the tension capacity (C_T) of strands, statistical expressions for compressive strength of concrete, prestress losses in strands, void and damage conditions of PT systems, and principles of structural mechanics.
- Moment demand at mid-span can be modeled using statistical expressions for the dead loads and live and impact loads due to HS20 and HL93 loading conditions.
- Time-variant structural reliability can be assessed based on strength and service limit states.
- Time-variant strength reliability (i.e., based on ultimate bending moment) can be modeled and assessed using flexural capacity and demand models of PT bridge girders.
- Time-variant service reliability (i.e., based on in-service stresses on extreme fibers) can be modeled and assessed using the allowable and applied in-service stresses (i.e., stress capacity and demand, respectively) of PT bridge girders.

1.8. RESEARCH METHODOLOGY

A coupled experimental–analytical research methodology has been developed to attain the research objectives listed in Section 1.6. Figure 1-3 provides a schematic showing experimental and analytical programs of this research.

The experimental program includes electrochemical characterization of PT strands and systems. This is performed using the results from electrochemical testing: cyclic polarization tests, galvanic corrosion tests, strand corrosion tests, and wire corrosion tests. These electrochemical tests are shown in the shaded box in Figure 1-3.

The analytical program includes: 1) the development of probabilistic models for time-variant tension capacity of PT strands and 2) the modeling and assessment of time-variant structural reliability of PT bridges. The results from the strand and wire corrosion tests will be used to perform this analytical program, as indicated by the dashed box in Figure 1-3.

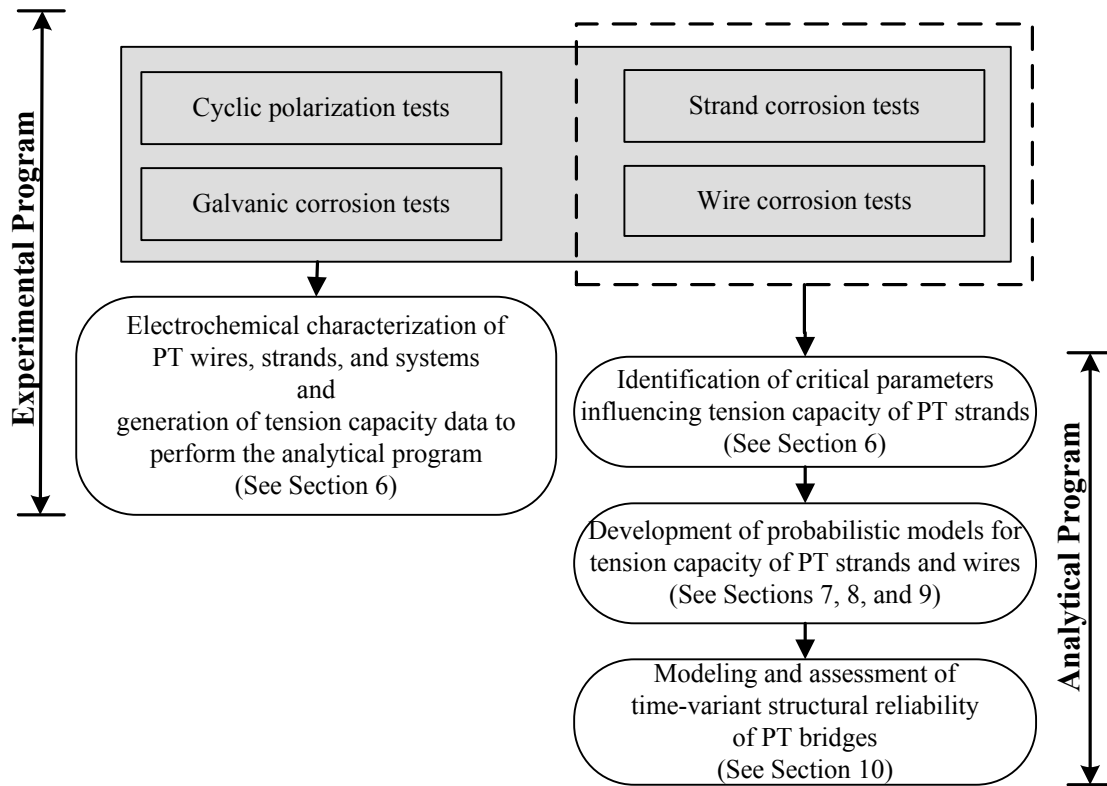


Figure 1-3. Schematic Showing Various Elements of Experimental-Analytical Program.

1.9. ORGANIZATION OF THE REPORT

This research report is divided into two volumes. This is Volume 1 and presents the characteristics of (PT) bridges and environmental conditions in Texas. Volume 1 also presents the electrochemical and C_T behavior of PT systems and the time-variant structural reliability of a typical PT bridge.

Volume 2 presents the literature review and experimental program and results on procedures to inspect voids and damages in PT systems, characteristics of PT grout materials,

and procedures to repair the voids in PT tendons. Volume 2 also presents the analytical models to assess economic risk associated with the mitigation strategies for PT bridges.

1.10. ORGANIZATION OF VOLUME 1

This document is organized using a section–subsection format. There are nine sections and several subsections within each section. The word “section” corresponds to the first heading level and “subsection” corresponds to the second, third, and fourth heading levels. Following is a brief explanation of the various sections in this report.

- Section 1 (the current section) starts with a brief discussion on prestressed concrete technology and its development. This is followed by discussions on key definitions used in this research. The research motivation and objectives are then presented. Following this, the assumptions and methodology of this research are provided.
- Section 2 provides a comprehensive review of literature. This section starts with a review of some case histories of failure of PT systems. A review of parameters influencing corrosion and modeling of corrosion process is then provided. Following this, the formulations for moment capacity, in-service stresses, and bridge load demands are reviewed. Then modeling and assessment of structural reliability are reviewed. Towards the end of Section 2, typical characteristics of segmental bridges in Texas are discussed.
- Section 3 presents the current research needs and the significance of this research.
- Section 4 presents the environmental characterization maps and corrosion risks of various geographic locations in Texas.
- Section 5 provides details of the experimental program. Details of all the materials used in the experimental program are provided first. The experimental design and test layout and other details of cyclic polarization, galvanic corrosion, and strand and wire corrosion test programs are then provided.
- Section 6 presents the experimental results from the cyclic polarization test, galvanic corrosion tests, and strand and wire corrosion tests. Also, critical parameters influencing strand capacity are identified in this section.

- Section 7 presents the probabilistic models for the C_T of strands under WD exposure conditions. This section also presents the statistical procedures used to develop probabilistic models for C_T of strands.
- Section 8 presents the probabilistic models relating the C_T of strands and wires under WD exposure conditions.
- Section 9 presents the probabilistic models for the C_T of strands under WD and CA exposure conditions.
- Section 10 presents the development of strength and service reliability models of PT bridges. As an application of these models, this section also presents the time-variant structural reliability of a typical PT bridge at various exposure conditions.
- Section 11 presents conclusions from this research. Recommendations for future research initiatives are also presented in this section.

2. LITERATURE REVIEW

Equation Section (Next)

2.1. INTRODUCTION

This section starts with the review of existing literature on various deterioration and failure cases of PT bridges. Parameters affecting strand corrosion mechanisms are then reviewed. Following this is a review of procedures and principles to determine the flexural capacity and demand of PT bridges. This is followed by a review of structural reliability assessment methods. Typical material, geometrical, and design characteristics of PT bridges in Texas are then reviewed.

2.2. DETERIORATION OF POST-TENSIONED BRIDGES

Most PT bridges are structurally healthy. However, due to poor construction and design practices, cases have occurred with the failure of one or more tendons, collapse of the entire PT bridge superstructure, or both. A review of failures of bridges with internal and external PT systems is provided next.

2.2.1. Failures of internal, grouted, post-tensioned systems

2.2.1.1. *Bickton Meadows Footbridge, UK*

The collapse of Bickton Meadows footbridge (located in Hampshire, UK) in 1967 was the first significant corrosion-related failure case of bonded PT bridges (NCHRP 1998). The precast segments in this bridge were jointed with mortar. These mortar joints were thin and highly permeable and hence allowed moisture, chlorides, and oxygen to be easily transported towards the steel tendons that cross the joints. This resulted in accelerated corrosion of steel tendons. At the time of collapse, the bridge was 15 years old.

2.2.1.2. *Ynys-y-Gwas Bridge, West Glamorgan, UK*

The Ynys-Y-Gwas Bridge, West Glamorgan, UK, was a segmental, PT, I-beam bridge spanning across the Afan river (Figure 2-1). This bridge had a clear span of 60 ft (18.3 m) and was constructed in 1952. The construction was based on a proprietary design with nine longitudinal beams supporting the deck. These beams were made of segmental I-sections and box sections and were held together using five longitudinal prestressing cables. Each cable consisted of 12 0.2-inch (5 mm) diameter high tensile steel wires. In 1985 (i.e., after 33 years of service), this bridge collapsed (Figure 2-1) under self-load and without warning.



Figure 2-1. Collapsed Ynys-y-Gwas Bridge (YouTube 2009).

A post-disaster study conducted by Transport and Road Research Laboratory (TRRL), UK, reported the following findings (TRRL 1987):

- Corrosion of the longitudinal tendons near the joints between the segments was the main cause for the bridge failure.
- The chlorides and moist environment under the bridge seem to be the major causes for corrosion of wires.
- The 1-inch (25 mm) wide transverse joints between the segments were filled with in-situ dry mortar caulking, through which moisture and chlorides easily infiltrated.
- Chlorides, possibly from de-icing salt, were present in the grout collected from joints between the segments (but not in the grout collected from the regions within the segments) and in the corrosion products.

- Large voids were found in ducts, but wires were covered with a layer of cement paste.
- Exposed wires in partially grouted ducts were severely corroded.
- No corrosion was found in fully grouted tendons.

2.2.2. Failures of external, grouted, post-tensioned systems

Although rare, some early-age tendon failures on external, grouted PT bridges in Florida, a state with 72 PT bridges, and Virginia raise concerns about the durability of such bridges, especially when exposed to moist and saline environments. This section provides a review of the tendon failures on the Niles Channel, Mid-Bay, Bob Graham Sunshine Skyway, and Varina-Enon bridges.

2.2.2.1. Niles Channel Bridge, Florida, US

The Niles Channel Bridge (Figure 2-2) is part of the Overseas Highway in the Florida Keys and is a 7-mile long, low-level, precast, segmental bridge with 234 external, grouted tendons. The bridge was constructed in 1983, and after 16 years of exposure to saltwater, one of the tendons near an expansion joint failed (FIB 2001). This failure was due to severe corrosion caused by the infiltration of chloride-contaminated water into the tendon anchorage that contained voids (FIB 2001, FDOT 1999). This failed anchorage was located at an expansion joint. A Florida Department of Transportation study (FDOT 1999) recommended the inspection and “intense scrutiny” of all the tendons adjacent to open expansion joints. Also, recommendations were made for the visual inspection and chloride analysis of pourback material at anchorages (cementitious material covering the face of the tendon anchorage) to assess the corrosion vulnerability of the tendon systems.



Figure 2-2. Niles Channel Bridge, Florida Keys, Florida (Structurae 2009).

2.2.2.2. *Mid-Bay Bridge, Florida, US*

The Mid-Bay Bridge in Choctawhatchee Bay, Florida (Figure 2-3), is another low-level, precast, segmental bridge with each span typically consisting of five 18-ft (5.5 m) segments. A routine inspection in 2000 detected corrosion-induced failure of one external tendon and part of a second external tendon. Figure 2-4 shows typical images of this strand corrosion, exposed strand along a bleed water trail, and cracked PT ducts in this bridge. The degree of corrosion was so severe that a total of 11 out of 846 tendons had to be replaced on this bridge (FDOT 2001a).



Figure 2-3. Mid-Bay Bridge, Choctawhatchee Bay, Florida (Worldbreak 2009).

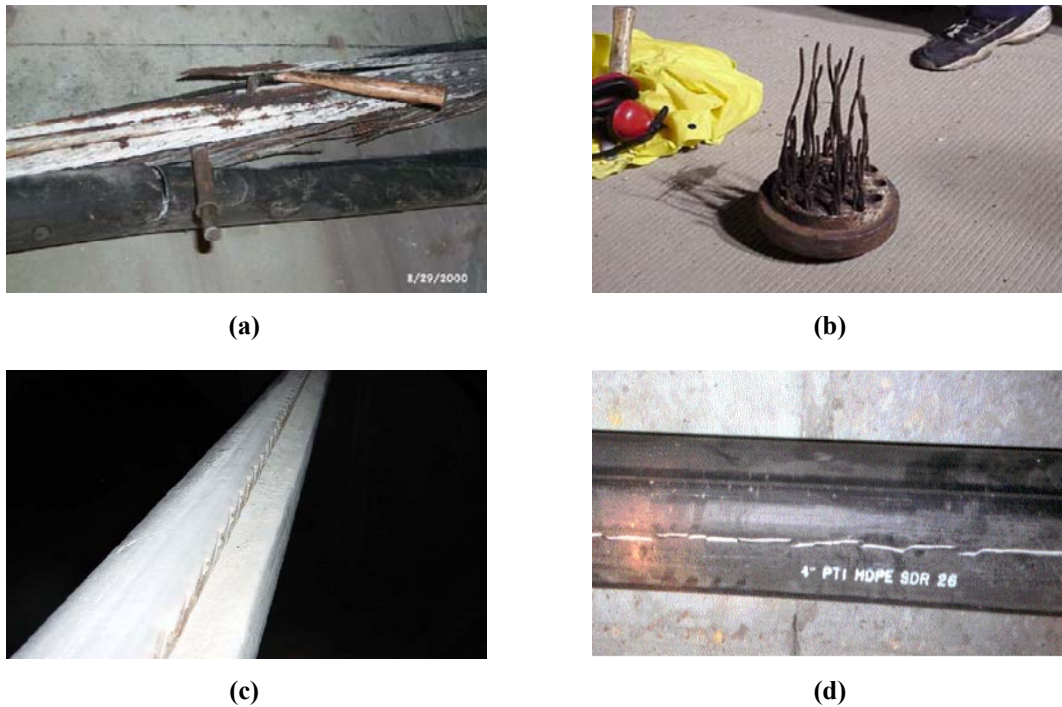


Figure 2-4. (a) Tendon Corrosion near Anchorage Zone, (b) Tendon Corrosion at Anchorage, (c) An Exposed Strand along a Bleed Water Trail, and (d) Cracked PT Ducts in the Mid-Bay Bridge, Florida (FDOT 2001a).

FDOT (2001a) reported the following major findings regarding the damages on this bridge:

- Severe tendon corrosion was found at and near anchorage zones.
- A large amount of voids (formed due to the formation, evaporation and/or re-absorption of bleed water) were found in the ducts at or near the anchorage zones and the midspan regions.
- The grout cover around the strands inside the ducts varied from 0 to 1.5 inches (38 mm).
- Because of the carbonation of the grout to a depth of approximately 0.25 inches (6 mm), the pH of the grout near the void regions was between 8 and 9 and the pH of the bulk grout inside was approximately 12. The color of the grout surface adjacent to the voids and the bulk grout varied from chalky white to dark grey. Also, a white powdery deposit, possibly the aluminum powder from an expansive agent, was observed on the grout surfaces near voids.

- Some HDPE ducts surrounding the tendon were cracked, possibly because the duct could not meet the construction specifications and/or resist the bending stresses. This excessive bending stress may have been the combined effect of the stresses developed due to the setting of grout at low temperature, the expansion of grout with expansive agent, and the presence of voids.

2.2.2.3. *Bob Graham Sunshine Skyway Bridge, Florida, US*

On May 9, 1980, *Summit Venture*, a large freight ship, collided and damaged the piers of the old Sunshine Skyway Bridge, which was located in Tampa Bay, Florida. The bridge was demolished, and a new bridge, the Bob Graham Sunshine Skyway Bridge (Figure 2-5), was later constructed and commissioned in 1987. The cost of the structure was \$244 million. Later, this 4-mile long, segmental, cable-stayed, PT bridge won many awards for its design and construction. However, within 8 years after construction, the top and bottom anchorage zones of the vertical tendons in the new bridge piers experienced severe corrosion damage, as shown in Figure 2-6.



Figure 2-5. Bob Graham Sunshine Skyway Bridge, Tampa Bay, Florida (Wikipedia 2009).

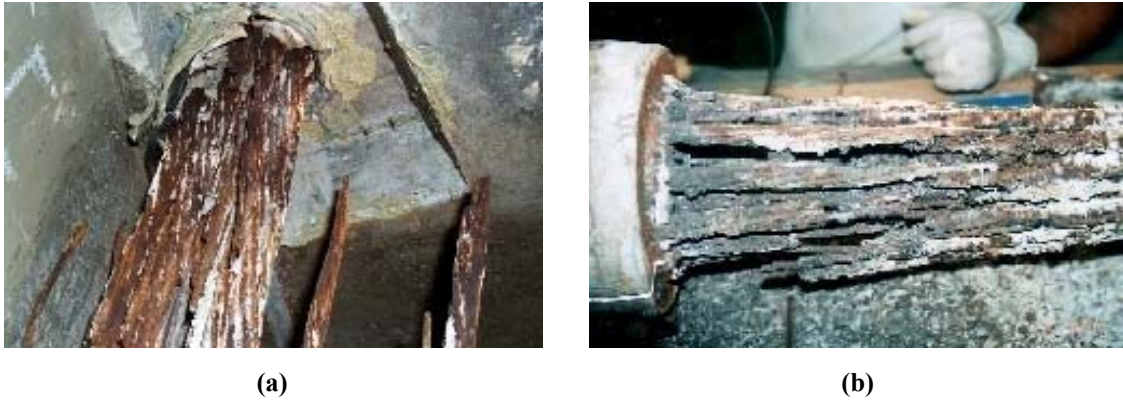


Figure 2-6. Severely Corroded Strands at Anchorage Zones of PT Columns in the Bob Graham Sunshine Skyway Bridge, Tampa Bay, Florida (FDOT 2001b).

FDOT (2001b) reported that the major direct and indirect causes for the tendon failure were:

- poor grout quality and grouting practices;
- presence of voids, formed due to bleed water evaporation, inside PT ducts, especially near anchorage zones;
- cracked HDPE ducts;
- infiltration of saltwater or surface drainage water into the PT ducts through the poorly sealed joints between the segments, openings at anchorages, cracks on ducts, etc.; and
- moist and chloride-contaminated environment near the exposed strands.

2.2.2.4. *Varina-Enon Bridge, Virginia, US*

The Varina-Enon cable-stayed, post-tensioned, segmental concrete bridge over the James River near Richmond, Virginia (Figure 2-7), was opened to traffic in 1990. One of the external tendons in this bridge failed in 2007 (after 17 years of service). Figure 2-8 shows the failed tendon. This tendon failure draws special attention because of the fact that the failure occurred at approximately 3 to 4 years after the voids in the tendon/duct were grouted or repaired. This repair work included filling voids inside the ducts using a high-performance grout. Hansen (2007) reported that the location of tendon failure was at the interface between the existing Class A or B grout and newer Class C repair grout. This failure indicates a critical need

for research into the influence of new repair grouts on tendon corrosion, especially at the interface between existing and repair grouts. This interface could possibly generate a galvanic corrosion cell. However, the validity of this argument needs to be investigated. It was also observed that many tendons were not completely sealed during the original construction. This likely enabled moist air from the environment to enter the voided ducts and accelerate corrosion of the exposed tendons.



Figure 2-7. Varina-Enon Bridge, over the James River, Virginia (Roadstothe future 2009).

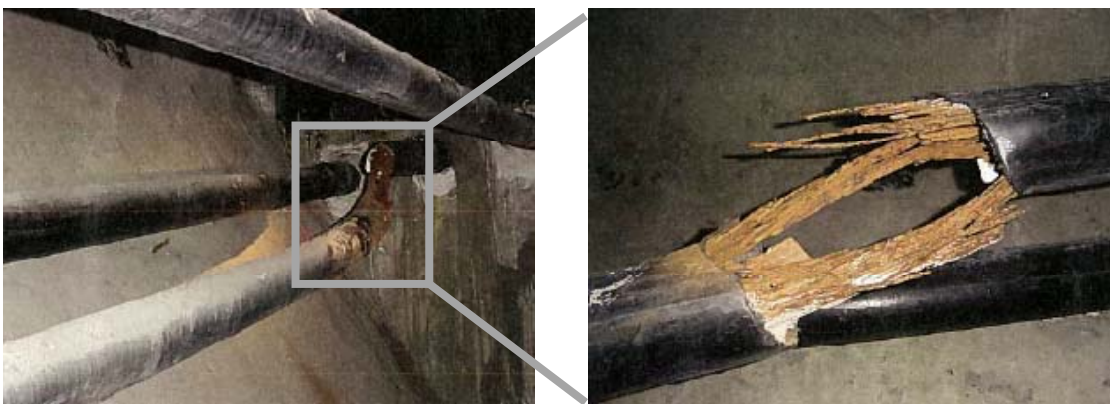


Figure 2-8. Tendon Corrosion in Varina-Enon Bridge, Virginia (Hansen 2007).

2.3. PARAMETERS INFLUENCING CORROSION OF POST-TENSIONED SYSTEMS

This section discusses the mechanisms of and parameters influencing corrosion of strands in PT bridges.

2.3.1. Dissimilar metallic materials in post-tensioned systems

In general, PT systems can consist of a variety of dissimilar metals. These metals can include ASTM A 416 (prestressing strands), ASTM A 615 (spiral bursting reinforcement), ASTM A 536 (cast-iron bearing plate or trumpet), ductile iron trumpet, ASTM A 53 (mild steel pipe), galvanized steel ducts, and others. The corrosion of any of these metallic materials can cause significant issues in load transfer mechanisms of PT systems. These dissimilar metallic materials are sometimes in close vicinity of each other. Such a situation, with or without the presence of other corrosive conditions, can initiate galvanic corrosion. According to Jones (1996), cast iron in seawater is more noble than steel in seawater. FDOT (2002) investigated galvanic corrosion activity (under moist and chloride environments) between ASTM A 513 mild steel pipe (metallurgically comparable to the ASTM A 53 mild steel pipe used in column foundations of the Bob Graham Sunshine Skyway Bridge), ASTM A416 strands, and ductile iron bearing plates. Figure 2-9 shows pitting corrosion on the ASTM A 53 Grade 5 pipe and ASTM A 416 strand due to coupled galvanic- and chloride-induced corrosion.



Figure 2-9. Pitting Corrosion on ASTM A 53 Pipe and ASTM A 416/A 416M-99 Strand Due to Coupled Galvanic- and Chloride-Induced Corrosion (FDOT 2002).

The major findings of the FDOT (2002) study are summarized as follows:

- The electrochemical potentials of both mild steel pipe and strands were approximately -575 millivolts versus a Cu/CuSO_4 electrode. However, visual observations and electrical measurements on the pipes and strands indicated that the pipe is more anodic to the strands.
- The electrochemical potential of the strand was approximately 100 mV more negative (i.e., more anodic) than that of bearing plate.
- The rate of the chloride- and moisture-induced corrosion of the strands was higher than the rate of galvanic-corrosion.
- If the dissimilar metals and their surroundings remain dry, it is less likely for galvanic corrosion to occur.

Although the FDOT (2002) study found that galvanic corrosion is not a concern if the PT systems are kept dry, it is practically impossible to ensure that the moisture and chlorides do not contaminate the PT systems. Hence, Sanchez et al. (2003) recommended that additional studies be performed on galvanic corrosion of PT concrete systems exposed to moisture and chloride environments. This recommendation was based on the corrosion stains observed on conventional reinforcement at or near anchorages on PT bridges in Texas.

2.3.2. Grout class

The Post-Tensioning Institute (PTI) classifies grout materials into four classes based on material specifications and field requirements (PTI 2003). Class A grout consists of cement and water only. Class B grout consists of portland cement, water, and mineral/chemical admixtures. Class C grout consists of pre-packaged, engineered grout material and water only. Class D grout is defined as any “special” grout designed by the engineer.

In general, Class A or B grout was used for most PT bridges constructed before the year 2000. Many research reports (ASBI 2000, FDOT 2001a, FDOT 2001b) indicate that Class A or B (non-thixotropic) grouts result in more bleed water and, therefore, a greater number of voids and cracks than Class C (thixotropic and low-bleed) grouts. Hence, given that other conditions remain the same, the tendons with Class A or B grouts will likely have more voids or crack

locations with accelerated corrosion than tendons with Class C grout. This may result in a higher joint probability of failure of tendons with Class A or B grouts than those with Class C grout. For these reasons, Class C grout is being used in most new PT bridge constructions that started after 2000 (Hansen 2007). In short, Class A or B and Class C grouts have been used in most PT bridges in the US.

Many Class C grouts with good flow characteristics are commercially available and being used in PT bridges. Three major commercially available Class C grouts are SikagROUT 300PT[®], Masterflow[®] 816, and Euco Cable grout PTX. As mentioned before in Section 2.2.2.4, although not within the scope of this research, the corrosiveness of these and other emerging new grout products needs to be investigated, especially when used as a repair material.

2.3.3. Voids

Many research reports (FDOT 1999, FDOT 2001a, FDOT 2001b, NCHRP 1998) cite voids as one of the major causes for corrosion of strands in PT systems. Review of these documents indicates that the corrosion is more severe at inclined and vertical anchorage zones than at horizontal locations on tendon systems. Figure 2-10 shows elevation views with typical void conditions at the anchorages in a PT column and at the anchorage, deviator block, and midspan regions in PT girders. Earlier, Figure 1-2 showed typical cross-sectional views of these tendons with no, partial, and full voids.

In addition to the increased degree of corrosive exposure conditions at anchorage zones with voids, the geometry of the interface between grout, air, and strands (GAS interface) at these voids may also influence the rate of corrosion. This is based on the well-known fact that the rate of electrochemical corrosion at an interface between a metallic and non-metallic material can be different from that at non-interface on the same metallic material. In the case of the GAS interfaces in tendons, the changes in corrosion rates can be due to the differential pH and other interfacial characteristics. These corrosion rates also depend on the cathode-to-anode ratios at these interfaces.

The size, number, and distribution of voids in a PT duct can be different at different locations and depend on global tendon profiles, local strand profiles, material characteristics,

grouting techniques, design considerations, and other factors. Due to limited accessibility, the difficulty in determining the continuity of voids, and other issues, it is difficult to measure the number and size of voids in tendons. However, it can be presumed that the size of critical voids can range from small to large volumes.

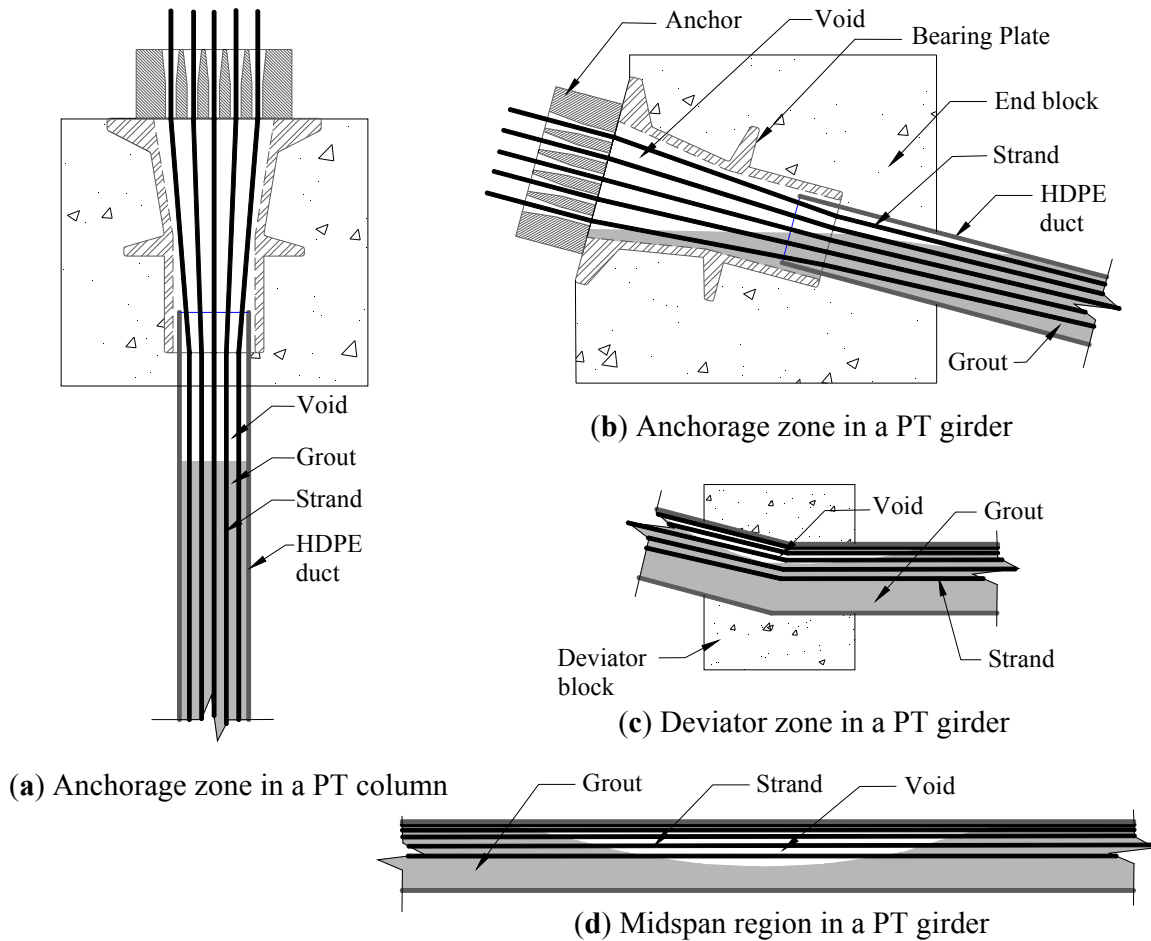


Figure 2-10. Elevations Showing Typical Void Locations in Grouted Tendon Systems.

Woodward (1981) reported that more than 50 percent of ducts in 12 bridges constructed between 1958 and 1977 contained voids with sufficient size to expose the tendon and accelerate corrosion. An inspection performed in the early 1970s of a segmental, PT bridge constructed in 1961 reported that 62 percent of the internal ducts contained voids that were continuous along

the whole length of tendons. The same inspection found that two ducts contained no grout (Woodward et al. 2001). Later, more and more voids and void-induced corrosion were detected in approximately a dozen PT bridges worldwide. TxDOT (2004) provides information on the percentage of voids present in the anchorage and along the tendons in the San Antonio “Y” bridge. A summary of this information is provided in Table 2-1.

Table 2-1. Percentage of Voided Tendons among the External Tendons on San Antonio “Y” Bridge (Adapted from TxDOT [2004]).

Test location	Number of tests performed	No. of voids detected	Percentage of voids detected
At or near anchorage zones	601	462	78.6
Near midspan regions	2963	86	6.6

2.3.4. Oxygen concentration

The presence of oxygen (O_2) is necessary for the occurrence of electrochemical corrosion of steel embedded in cementitious materials (ACI 2003). In general, the normal atmospheric oxygen level is 21 percent. However, because box girders can be considered as confined spaces, and if the steel tendons are covered by cementitious grout and encased by high density polyethylene (HDPE) ducts with low air permeability, the level of oxygen to which the strands are exposed can be lower than 21 percent. Still, the presence of many cracks on the HDPE ducts and other openings in the tendon and anchorage systems have been reported (FDOT 1999, FDOT 2001a, FDOT 2001b). In these cases, a sufficient amount of oxygen can infiltrate the ducts to facilitate the electrochemical process. Less oxygen availability may be one reason why there is less corrosion near the midspan regions than at the anchorage regions of a PT duct with no cracks.

2.3.5. Cementitious pore solution and pH

In bridge structures, the metallic reinforcing materials are embedded in cementitious materials. For example, the immediate environment of the steel embedded in concrete and grout is the pore solution (liquid phase). Diamond (1981) discovered that the grout pore solution contains Na^+ ,

K^+ , and Ca^{++} ions, and the concentration varies during the initial curing period but stabilizes with the time of hydration (Figure 2-11).

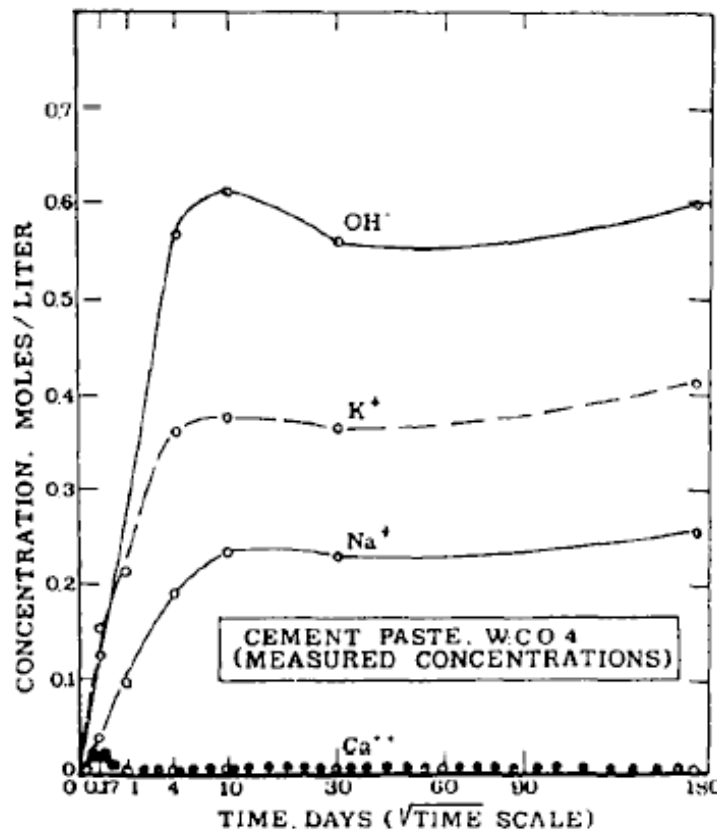


Figure 2-11. Variation of Pore Solution Composition with Time of Hydration (Diamond 1981).

Several researchers have studied the pore solution chemistry. Diamond (1981) and Cui and Sagues (2003) provide the chemical composition for the laboratory test solutions to simulate the actual grout pore solution for electrochemical tests. As in the case of uncontaminated grouts, the pH of these simulated pore solutions is very high (say, approximately 13.5), indicating high alkalinity. Neville (1998) and others indicate that steel is resistant to corrosion when exposed to a highly alkaline pore solution, and as the pH drops (say, due to carbonation or other mechanisms) the steel becomes more vulnerable to corrosion.

2.3.6. Carbon dioxide concentration

The presence of carbon dioxide (CO_2) in the atmosphere also poses a challenge for corrosion of steel reinforcement embedded in cementitious materials. When carbonated, the pH of the cementitious material can drop from between 12.6 to 13.5 to as low as 9 (Neville 1998). Because of this drop in the pH pore solution, the embedded steel reinforcement actively corrodes (ACI 2003). Hence, the corrosion rate of strands, when exposed to carbonated grout, could be useful in predicting the time-variant structural reliability.

According to a study by the American Concrete Institute, (ACI 2003), the CO_2 level under normal atmospheric conditions is 0.03 percent. However, the CO_2 level to which the PT box girders are exposed may vary due to the exhaust gases from vehicles, industrial chimneys, or both. This indicates that a PT bridge in an urban or industrial area can have a higher rate of carbonation than one in a rural area. Sagues (2003) reported the carbonation of surface grout as a possible reason for localized corrosion at the GAS interface (Figure 2-12). According to an FDOT (2001a) report, the grout in a PT tendon on the Mid-Bay Bridge was carbonated to a depth of approximately 0.25 inches (6 mm), resulting in a drop in the pH of grout near the void regions from approximately 12 to a value between 8 and 9.

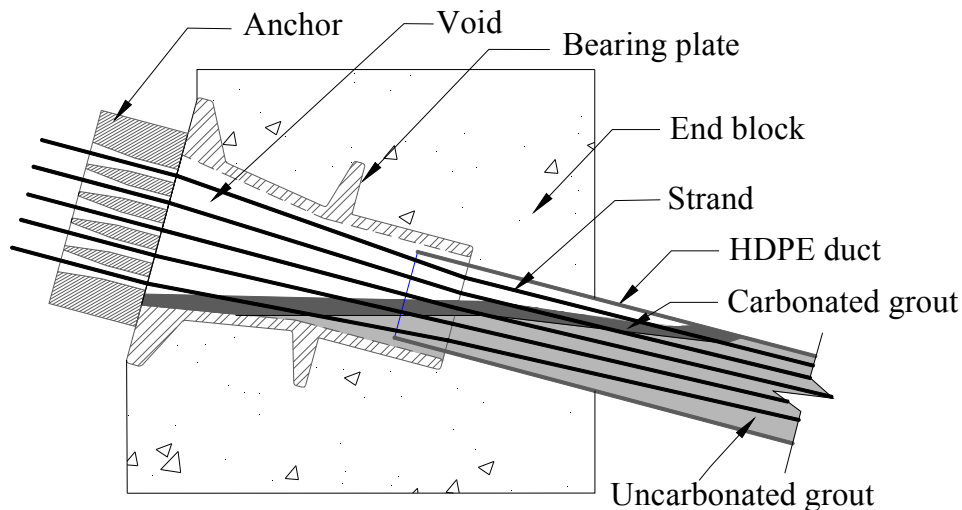


Figure 2-12. Carbonated GAS Interfaces in Anchorage Zones.

2.3.7. Moisture conditions and precipitation

It is well known that moisture conditions (MC) at the metal surface are an essential component for electrochemical processes of steel embedded in cementitious materials (ACI 2003). Woodward et al. (2001) reported the presence of water in internal tendons of a segmental bridge constructed in 1961. According to FDOT (2001a) and Hansen (2007), the infiltration of moisture into the PT ducts can occur through cracks or openings on HDPE ducts and at anchorage zones, especially at or near construction joints. Once water collects inside the ducts, it may never evaporate or escape from the duct system, unless the moisture is dried due to self-desiccation of the cementitious grout or artificial drying methods (e.g., passing dry air or inert gases through PT ducts). Perret et al. (2002) reported that the self-desiccation characteristic is commonly found in highly impermeable cementitious grouts, and Sagues (2003) identified this as the case with grouted tendons in PT bridges. There may also be cases with winter–summer (rainy–sunny) seasons that may induce an annual WD condition inside the PT ducts. However, the literature typically does not provide information on the history of moisture conditions inside PT ducts.

Direct exposure to WD conditions can create very severe corrosive conditions. Fortunately, such conditions are not predominant in many of the PT bridges that are located away from large bodies of water and protected from rainwater infiltration. However, FDOT (2001b) reported that cyclic charging (i.e., a WD condition) occurred on a bottom portion of a vertical tendon system and seawater leaked through joints between the segments and other openings. This caused failure of the tendons in the Bob Graham Sunshine Skyway Bridge.

A history of daily precipitation can provide an indication or assist in making an engineering judgment about the possible WD conditions that might exist inside a PT duct. The history of daily precipitation in San Antonio, Texas, in the years 2001 through 2005 is presented in Figure 2-13.

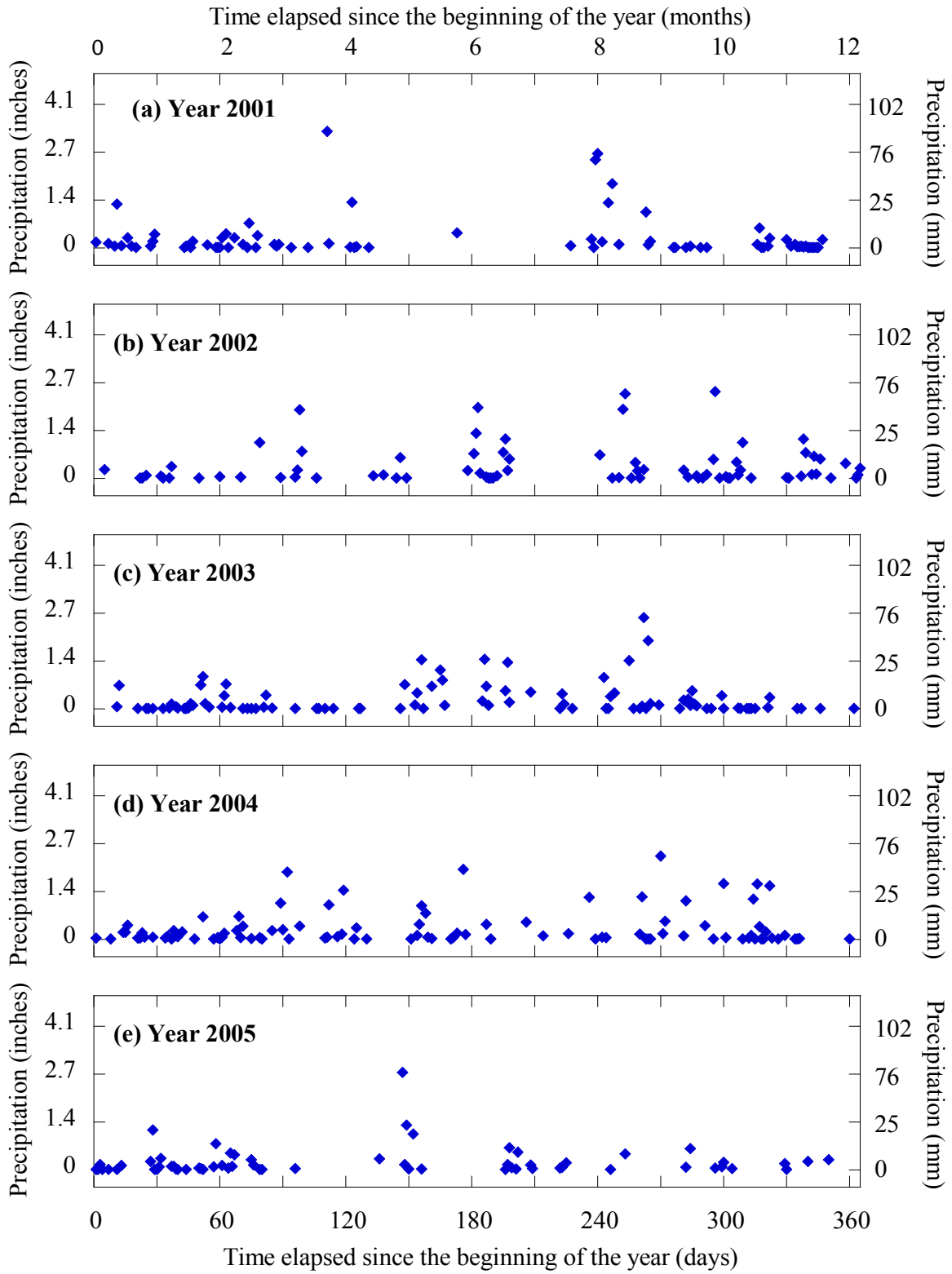


Figure 2-13. Daily Precipitation in San Antonio, Texas (NCDC 2009).

2.3.8. Time of wetness

For atmospheric corrosion of steel specimens, many researchers (Guttman and Serada 1968, Klinesmith et al. 2007) used Time of Wetness (*TOW*) as a parameter to model atmospheric corrosion of steel specimens. In these studies and per ISO 9223 designation, the *TOW* is defined as the time when the relative humidity (*RH*) is greater than 80 percent and temperature is greater than 0°C (32°F). Corvo et al. (2008) found that *TOW* is dependent on the presence of atmospheric pollutants, metal surface cleanliness, rain, dew, and water adsorption on metal surfaces. They do not recommend the use of *TOW* for predicting atmospheric corrosion under indoor and contaminated environments. Moreover, if *TOW* is used in corrosion modeling, the influence of contaminants and their interaction with *TOW* has to be established. Per ISO 9223 designation, Dean and Raiser (2002) assumed or approximated *TOW* as the time when *RH* was greater than 80 percent and the temperature (*T*) was greater than 32°F (0°C). Because of the model errors associated with such approximations and the *TOW*'s dependence on other exposure conditions, it would be more useful for practicing engineers to use *RH*, instead of *TOW*, to predict metal loss due to corrosion.

2.3.9. Relative humidity

A necessary requirement for atmospheric corrosion is the formation of a thin film electrolyte. In general, the higher the *RH* level, the greater the chances of formation of this thin film. According to Roberge (1999), there exists a critical *RH* level, defined as the minimum *RH* level needed to form this thin film. This critical *RH* level depends on the corrosion characteristics of the metallic surface, the tendency of corrosion products and surface deposits to absorb moisture, and the presence and type of atmospheric pollutants. Duncan and Ballance (1988) found that the presence of hygroscopic salts in the environment can lower the critical *RH* level. They reported that the critical *RH* level for a metallic surface contaminated by $\text{MgCl}_2 \cdot 6\text{H}_2\text{O}$ is 34 percent and NaCl is 77 percent. Many PT bridges are exposed to these salts in the form of airborne chlorides, seawater, or de-icing/anti-icing salt deposition.

In short, *RH* is a parameter that influences the rate of corrosion of steel surfaces exposed to atmospheric conditions. Haynie and Upham (1974) and Mansfeld (1979) indicated that as *RH*

increases, especially when 60 percent, the corrosion loss increases exponentially. The corrosion rate of an exposed tendon or strand at a void location could be related to the *RH* conditions inside the PT duct. Hansen (2007) cited moist air that infiltrated into the ducts through the cracks and openings in the tendon system as a cause for strand corrosion. Given that ducts can be cracked and openings can exist at anchorages, the *RH* at a void region inside PT ducts might be correlated to seasonal climatic conditions or proximity of the bridge to a water body. However, no mathematical relationship has been developed. Research addressing this issue could help to predict the long-term performance of the majority of PT bridges (i.e., those that are not exposed to aggressive corrosive conditions) and develop an appropriate repair and maintenance strategy. However, development of such a relationship is not within the scope of this research.

2.3.10. Chloride concentration

Corrosion can be significantly accelerated in the presence of water and chlorides. When available in sufficient quantities at the steel surface, chlorides act as a catalyst for the electrochemical corrosion of steel embedded in cementitious material. Trejo and Pillai (2003) determined the critical chloride threshold, defined as the minimum amount of chlorides required to initiate corrosion, for ASTM A615 steel embedded in a plain cement mortar to be 0.9 lb/yd³ (0.5 kg/m³, 0.09 weight % cement). However, there exists a large scatter in reported values for mild steels.

This literature review could not find any significant information on the critical chloride threshold of prestressing steel strands. Mammoliti et al. (1996) and Pillai and Trejo (2005) reported that, in general, the smoother the metallic surface, the higher the critical chloride threshold. However, FHWA's report that the critical chloride threshold of prestressing strands is approximately six times higher than that of the mild steel provided limited data to justify this finding, and further experimental work is needed.

NCHRP (1998) and FDOT (1999, 2001a and b) reported the presence of a high amount of chlorides in the tendons on the Bob Graham Sunshine Skyway Bridge. In general, the waterborne or airborne chlorides can infiltrate into the ducts and reach the strand surfaces, especially when the bridge is located above seawater and/or exposed to de-icing/anti-icing salts.

2.3.11. Temperature

Electrochemical reactions are influenced by the temperature of the metal surface. The higher the temperature, the greater the molecular energy levels and collision rates and, hence, electrochemical reaction rates will be. The Arrhenius equation can be used to represent the main effect of temperature on the corrosion rate constant, k , as follows:

$$k = A \cdot \exp\left(-\frac{E_a}{RT}\right) \quad (2.1)$$

where A is a pre-exponential factor, E_a is the activation energy of the reaction process, R is the Universal Gas Constant ($8.314472 \text{ JK}^{-1}\text{mol}^{-1}$ [$6.132440 \text{ lbf}\cdot\text{ft}\cdot\text{K}^{-1}\cdot\text{g}\cdot\text{mol}^{-1}$]), and T is the absolute temperature (in $^{\circ}\text{K}$).

The main effect of temperature on strand corrosion may be minimal or negligible during normal atmospheric temperature conditions. However, even in this temperature range, an interaction between the temperature, relative humidity/moisture, and chloride conditions could possibly result in a significant corrosion rate.

2.3.12. Axial stress

According to ACI (2003), the stress level does not play a major role in the corrosion susceptibility of conventional steel reinforcement (meeting ASTM A615 specifications) embedded in cementitious materials. PT strands (meeting ASTM A416 specifications) experience very high axial stresses (f_{axial}). The f_{axial} on PT strands could be approximately four times more than that experienced by ASTM A615 steel in bridges, and the influence of f_{axial} on corrosion could be significant. It should also be noted that conventional reinforcement is hot-rolled and PT strands are cold-rolled. The very high axial stress levels and the cold-rolled surface could influence the corrosion rate and susceptibility. Naaman (2004) reported that the average in-situ stress of a PT strand can be assumed to be equal to $0.545 f_{pu}$ or 147 ksi (1014 MPa). Proverbio and Longo (2003), Kovač et al. (2007), and Sanchez et al. (2007) reported that the synergistic effect of high stress levels and corrosive mediums can

influence corrosion susceptibility, especially stress corrosion cracking of prestressing strands. Also, Kovač et al. (2007) reported that cold-rolled, prestressing steel has a non-uniform microstructure with no cracks when f_{axial} is 0 and transgranular cracks (with typical crack length equal to approximately 100 μm) when f_{axial} is approximately $0.6f_{pu}$. These have not been observed in conventional steel reinforcement. Smaller transgranular cracks could be formed at stress levels lower than $0.6f_{pu}$. However, sufficient information on these surface cracks in PT strands and their effects on corrosion is not available in the literature. The difference in processing and finishing of conventional reinforcement and PT strands can have a significant effect on their corrosion performance.

2.3.13. Other factors

Fatigue loading could cause serious durability issues for bridge systems. Wollmann et al. (2001) performed live load tests (using AASHTO HS20 trucks) on several spans of the San Antonio “Y” bridge and found that the f_{axial} varies less than 7 Mpa (1 ksi). Because this is much less than the 70 Mpa (10 ksi) criteria set by Ryall et al. (1993), it can be concluded that fatigue may not be a significant durability issue of typical PT bridges in Texas. However, a synergistic effect of fatigue and aggressive corrosive conditions might cause durability issues. Other concrete deterioration mechanisms such as alkali-aggregate reaction (AAR), delayed-ettringite formation (DEF), freeze-thaw cycles, and other mechanisms affecting the integrity of hardened concrete can also create durability issues for PT bridges.

2.4. CORROSION EVALUATION USING VISUAL INSPECTION TOOLS

Visual observation of uncleaned, corroded strands by the unaided eye cannot generate reliable information on the degree, form, or pattern of corrosion, especially when the corrosion attack is not significant, does not vary significantly from one sample to another, or both. Sason (1992) recommended a strand cleaning method to help field engineers in “accepting” or “rejecting” new PT strands before placing inside the ducts. In this method, strands are hand-rubbed with a synthetic cleaning pad until the loose corrosion products or rust is removed. Because no mechanical pressure is applied and the pad is non-metallic, it will not remove the base metallic material from the strands. Sason (1992) also reported, “In many cases, heavily rusted strands

with relatively large pits will still test to an ultimate strength greater than specification requirements. However, it will not meet the fatigue test requirements.” No definition was provided for “relatively large pits” and, hence, corrosion evaluation would heavily depend on the inspector’s experience, judgment, or both. Also, effective visual inspection of strands or tendons inside box girders could be very difficult and time consuming because of the large number of strands, insufficient lighting, opaque HDPE duct covering, and other factors. Although difficulties exist, direct inspection (using naked eyes) and indirect inspection (using photographs or micrographs) of cleaned strands can assist in identifying the presence of surface corrosion, pitting corrosion, corrosion at or near GAS interfaces, or other unique patterns that could influence the capacity of PT strands.

2.5. MODELING CORROSION OF STEEL UNDER IMMERSION CONDITIONS

Strands in PT bridges may experience WD conditions. Probabilistic models that predict the corrosion and/or capacity loss of strands exposed to WD conditions would be helpful to develop time-variant structural reliability models. Such models were not identified in this literature review. However, corrosion models for unprotected steel (i.e., bare steel) exposed to immersion conditions have been developed by Southwell et al. (1979) and later by Melchers (1998, 1999). These models were based on data collected from actual steel structures (e.g., ship hulls exposed to sea water) and are not necessarily suitable for the WD conditions inside a PT duct or tendon. Table 2-2 shows mathematical expressions and Figure 2-14 shows graphical expressions of these linear, bi-linear, tri-linear, and power models.

Table 2-2. Existing Models for Thickness Loss of Steel Surfaces on Ships Exposed to Seawater.

Model type	Existing wet-dry corrosion models	Reference
Linear	$K = 0.076 + 0.038t$	Southwell et al. (1979)
Bi-linear	$K = \begin{cases} 0.09t, & 0 < t < 1.46 \text{ years} \\ 0.076 + 0.038t, & 1.46 < t < 16 \text{ years} \end{cases}$	Southwell et al. (1979)
Tri-linear	$K = \begin{cases} 0.170t, & 0 \leq t < 1 \text{ year} \\ 0.152 + 0.0186t, & 1 \leq t < 8 \text{ years} \\ -0.364 + 0.083t, & 8 \leq t < 16 \text{ years} \end{cases}$	Melchers (1998)
Power	$K = 0.1207t^{0.6257}$	Melchers (1998)
Power	$K = 0.084t^{0.823}$	Melchers (1999)

K = corrosion loss (in mm); 1 inch = 25.4 mm

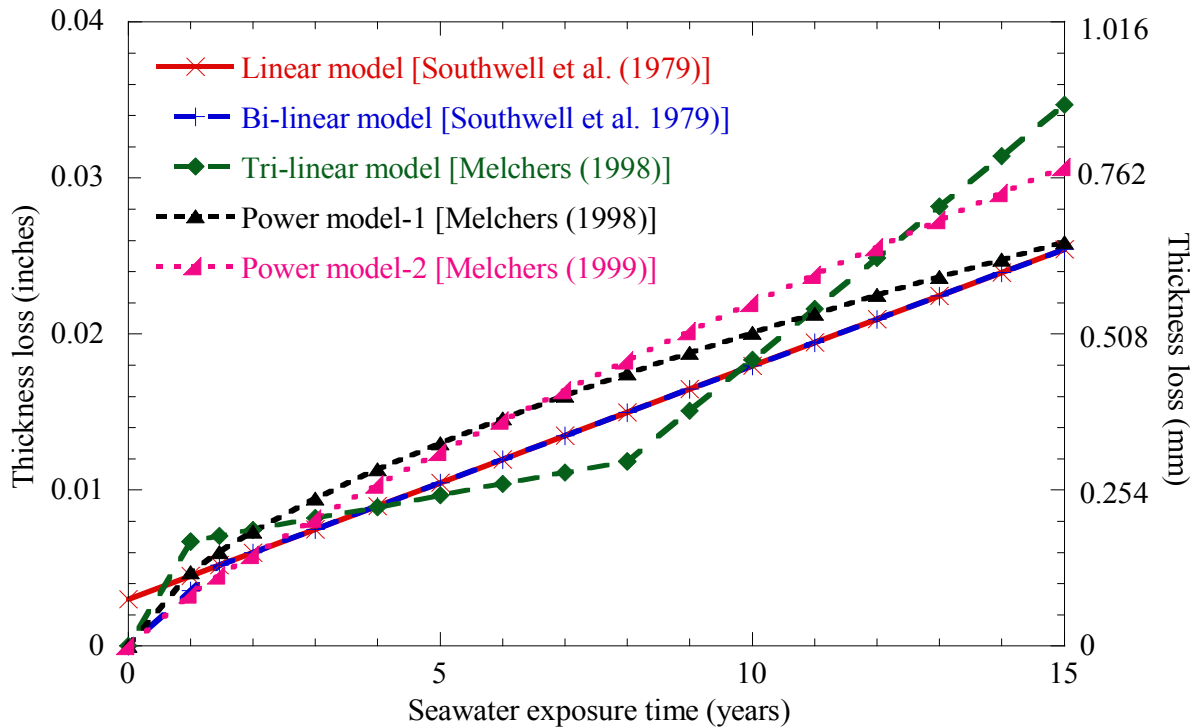


Figure 2-14. Thickness Loss of Steel Surfaces on Ships Exposed to Seawater (Predicted Using Existing Corrosion Models).

In Table 2-2, note that the linear model and the second part of the bi-linear models are identical. In Figure 2-14, the tri-linear model, with diamond data markers, underpredicts the thickness loss when time is between approximately 5 and 10 years and overpredicts the thickness loss after 10 years, when compared to those predicted by linear, bi-linear, and power models. Moreover, for practical engineering purposes, all these models tend to provide similar results up to 5 years, beyond which there are only slight variations. However, these models that are made for seawater exposure of ship structures cannot be used for predicting capacity loss of PT strands exposed to WD environments. This is mainly because of the different material, void, oxygen, moisture, and stress conditions of the PT systems.

2.6. MODELING CORROSION OF STEEL UNDER CONTINUOUS-ATMOSPHERIC EXPOSURE CONDITIONS

This subsection begins with a review of various atmospheric corrosion models as a function of environmental parameters and time. The model forms that are used to predict long-term corrosion are then discussed. Following this, the procedures to determine and modify the power corrosion model based on recent field data are provided. Various long-term corrosion models are then presented.

2.6.1. Atmospheric corrosion models based on environmental parameters, time functions, or both

A collection of atmospheric models that use environmental parameters, exposure time function, or both is provided in Table 2-3. The symbols in Table 2-3 are as defined by the corresponding authors and may or may not be representative of symbols used in the remainder of this document.

Table 2-3. Existing Atmospheric Corrosion Models (as a Function of Environmental Parameters and Exposure Time) for Steel.

Atmospheric corrosion models	Reference
$K = 0.16 TOW^{0.7} (SO_2 + 1.78)$ where, K = grams; TOW = days; SO_2 = ppm	Guttman and Serada (1968)
$K = 32.55 \cdot \sqrt{t} \cdot e^{\left(\frac{0.00275 SO_2 - 163.2}{RH}\right)}$ where, K = mm; t = years; SO_2 = mg/m ³ ; RH = %	Haynie and Upham (1974)
$K = 0.0152 \cdot TOW^{0.428} \cdot SO_2^{0.57}$ where, K = g/m ² /day; TOW = hours/year; SO_2 = mg/m ² /day	Barton et al. (1980)
$K = 1.17 \cdot TOW^{0.66} \cdot (SO_2 + 0.048)$ where, K = mm; TOW = hour/year; SO_2 = mg/m ² /day	Hakkarainen and Ylasaari (1982)
$K = 1.327 + 0.4313 SO_2 + 0.0057 TOW + 0.1384 Cl^-$ where, K = mm; TOW = days; SO_2 = mg/m ³ ; Cl^- = mg/m ² /day	Knotkova et al. (1984)
$K = 132.4 Cl^- (1 + 0.038T - 1.96TOW - 0.53SO_2) + 74.6TOW (1 + 1.07SO_2) - 6.3$ $K = 33.0 + 57.4 Cl^- + 26.6 SO_2$ where, K = μm; TOW = annual fraction; T = °C; SO_2 = mg/m ² /day	Feliu et al. (1993)
$K = 18.851 t^{1.0947} (Cl^-)^{0.4313} (RF / D)^{-0.2447}$ where, K = g/m ² ; t = months; Cl^- = mg/m ² /day; RF = mm; D = days	Corvo et al. (2005)
$K = 0.0157 \left(\frac{TOW}{3800}\right)^{0.15} \left(1 + \frac{Cl^-}{50}\right)^{0.22} (RH)^{2.3} e^{(-0.0019(T+20))}$ where, K = mm; TOW = hours/year; Cl^- = mg/m ² /day; RH = %; T = °C	Klinesmith et al. (2007)

Cl^- = Chloride concentration; D = Number of rainfall days; K = Corrosion loss; RF = Rainfall; RH = Relative humidity; SO_2 = Sulfur dioxide concentration; T = Temperature; TOW = Time of wetness.

In the models provided in Table 2-3, the time function is represented by either TOW or an exponent of time in years. As shown in Subsection 2.3.8, the accurate estimation of TOW is very complex (Corvo et al. 2008). Moreover, sometimes it is not practical to have a good estimate of the environmental parameters over a long duration. Therefore, the use of models in Table 2-3 for long-term prediction of corrosion is limited.

2.6.2. Atmospheric corrosion models based on time functions only

To predict long-term corrosion loss, researchers have developed simple corrosion models that have only exposure time as a parameter. These models are developed based on long-term exposure data collected from corrosion test sites/stations.

The International Standard Organization (ISO) designation 9224, *Corrosion of Metals and Alloys – Corrosivity of Atmospheres*, provides guidelines for atmospheric corrosion rates of metallic materials. This standard assumes the following equation:

$$\begin{aligned} K &= tr_{av} && ; 0 \leq t \leq 10 \text{ years} \\ K &= 10r_{av} + (t-10)r_{im} && ; t > 10 \text{ years} \end{aligned} \quad (2.2)$$

where t is the time of exposure (in years) and r_{av} is the average corrosion rate (in mm/year) during the first 10 years, and r_{im} is the steady-state corrosion rate (in mm/year). However, this steady-state corrosion rate assumption does not represent the decaying corrosion rate as observed in most atmospheric corrosion processes.

To represent the decaying corrosion rates, a power model form has been used by many researchers, including Haynie and Upham (1974), McCuen et al. (1992), Kobus (2000), Laco et al. (2004), Corvo et al. (2005), Natesan et al. (2006), and Klinesmith et al. (2007). The ASTM G101, *Standard Guide for Estimating the Atmospheric Corrosion Resistance of Low-Allow Steels*, also suggests a power model form for predicting thickness loss due to corrosion. This model form is mathematically expressed as follows:

$$\text{Total corrosion loss} = At^n \quad (2.3)$$

where A equals the corrosion loss in the first year of exposure, t equals the exposure time, and n is a constant. In 1989, Drazic and Vascic proposed another corrosion model based on a natural logarithmic approximation as follows:

$$\text{Total corrosion loss} = a + b \ln[t] \quad (2.4)$$

where a and b are constants and t is the exposure time in years. This model also conforms to the decaying corrosion rate. However, the power model form seems to be more appropriate and, hence, widely used by researchers for modeling atmospheric corrosion processes. A collection of existing power corrosion models, following the model form in Equation (2.3) for carbon steel under marine, industrial, and rural exposure conditions, has been provided in Table 2-4. Figure 2-15 (a), (b), and (c) show the predicted thickness loss (using these power models) for carbon steel under marine, industrial/urban, and rural exposure, respectively. Note that the highest thickness loss predicted by a model for industrial/urban exposure condition in Figure 2-15 (b). However, a general comparison between the prediction curves in Figure 2-15 (a) and (b) indicates that marine and industrial/urban atmospheric exposure conditions may, in general, result in approximately similar thickness losses. Also, note that the lowest thickness loss is observed or predicted on the rural exposure condition, as shown in Figure 2-15 (c). It should be understood that all these models were developed based on the data from widely varying geographical, environmental, and other corrosive conditions. However, a general conclusion that the atmospheric corrosion of steel follows a power model is reasonable.

Table 2-4. Existing Power Models for Atmospheric Corrosion of Carbon Steel.

No.	A	n	Exposure condition	Location	Exposure times	Reference	
1	50.44	0.5413	Marine	Kure Beach, NC, US (250 m from sea coast)	Last test at 7.5 years	McCuen et al. (1992)	
2	46.74	0.45		Barcelona, Spain (close to sea coast)	0-, 3-, 6-, 9-, and 12-month tests	Laco et al. (2004)	
3	58.57	0.65		Cudalore, India (50 m from sea coast)	Based on a 5-year test	Natesan et al. (2006)	
4	32.43	0.35		Nagapattinam, India (50 m from sea coast)			
5	115.04	0.50		Mangalore, India (130 m from sea coast)			
6	30.58	0.62		Goa, India (50 m from sea coast)			
7	107.74	0.49	Industrial-marine	Manali, India (SO_2 , Cl , NO_3 , sea coast)	Based on a 5-year test	Natesan et al. (2006)	
8	59.69	0.7356	Industrial/ Urban	Rankin, PA, US	Last test at 10 years	McCuen et al. (1992)	
9	72.11	0.4501		Bethlehem, PA, US			
10	53.82	0.5242		Columbus, OH, US			
11	51.24	0.3297		Newark, NJ, US	Last test at 16 years		
12	75.27	0.3105		Bethlehem, PA, US			
13	50.51	0.3586		Newark, NJ, US	Last test at 15.5 years		
14	58.00	0.3445		Kearny, NJ, US	Last test at 20 years		
15	73.50	0.21		Katowice, Poland	0-, 1-, 2-, 4-, and 6-year tests		Kobus (2000)
16	37.50	0.35		Warsaw, Poland			
17	22.37	0.635		Barcelona, Spain	0-, 3-, 6-, 9-, and 12-month tests		Laco et al. (2004)
18	44.31	0.29	Mumbai, India	Based on a 5-year test	Natesan et al. (2006)		
19	29.65	0.46	Surat, India				
20	36.14	0.5984	Rural	Saylorsburg, PA, US	Last test at 16 years	McCuen et al. (1992)	
21	27.50	0.44		Boreka Forest, Poland	0-, 1-, 2-, 4-, and 6-year tests	Kobus (2000)	
22	14.52	0.35		Mahendragiri, India	Based on a 5-year test	Natesan et al. (2006)	

ΔT = Thickness loss in μm ; A and n empirical constants.

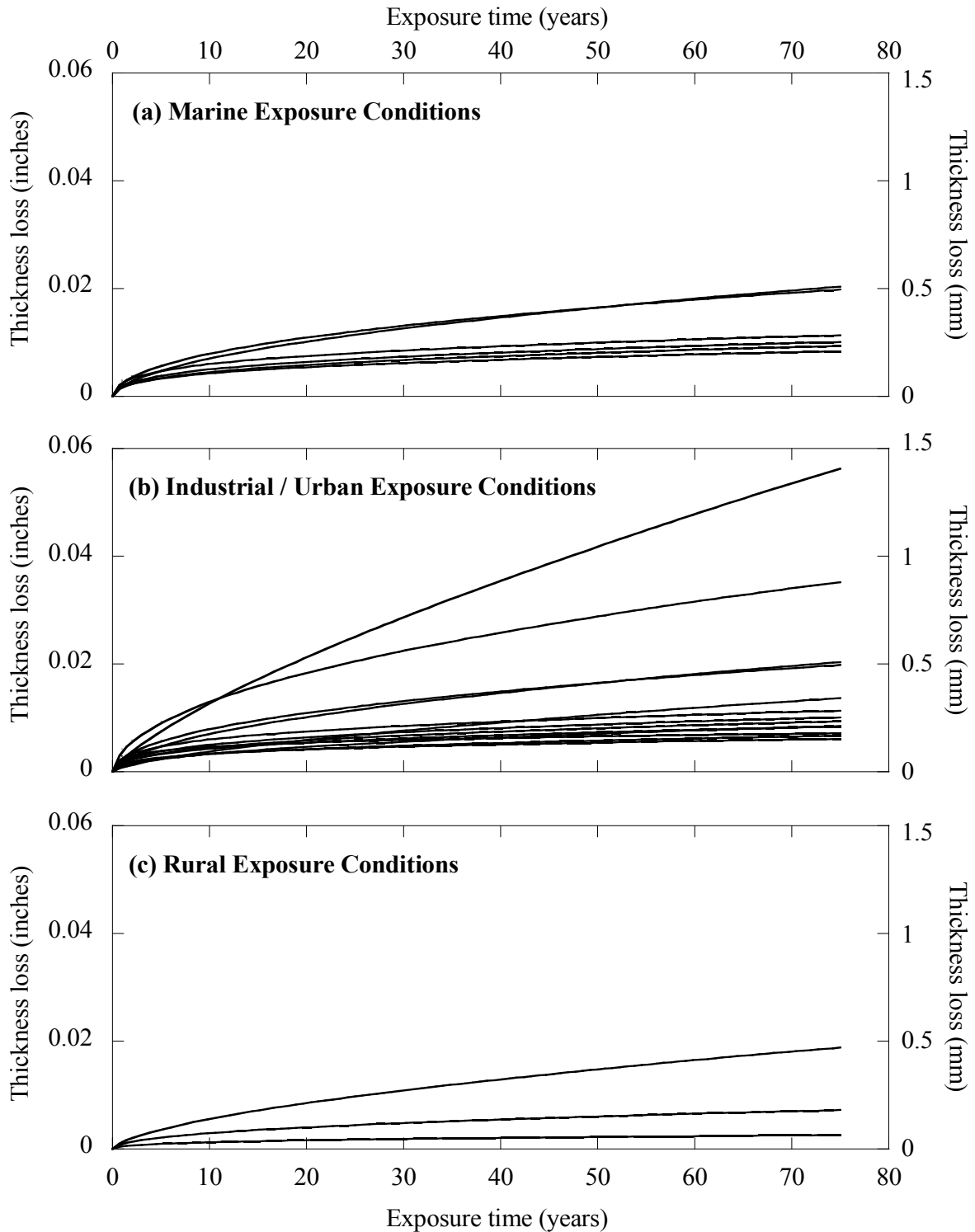


Figure 2-15. Thickness Loss of Carbon Steel Due to Atmospheric Corrosion (Predicted Using Existing Power Models) (Note: 1 inch = 25.4 mm).

In addition to predicting atmospheric corrosion, the power models have been used in underground corrosion modeling. For example, Decker et al. (2008) proposed the following equation for corrosion losses in steel piles embedded in soil:

$$\Delta T = 59 t^{0.67} \quad (2.5)$$

where ΔT is the average thickness loss (in μm) in t years of exposure. Romanoff (1962) proposed an exponent of 0.55 for predicting metallic corrosion in soils. In general, it can be concluded that the power model is very helpful in predicting long-term corrosion of various metal-electrolyte systems. In addition, the power corrosion model can be updated or modified based on new data available, possibly from field inspections. Following is a discussion on procedures for developing and then modifying/updating power corrosion models.

2.6.3. Developing and updating the power corrosion model

For the power corrosion model form in Equation (2.3), the initial estimate of A can be determined using the data from a short-term (say, 1 year) corrosion test. The initial estimate of n can be determined using the data from multiple times, if available. As evident from Table 2-4, the value of A and n can vary as the atmospheric exposure conditions change. Table 2-4 also indicates that the exponent term ranges between 0.21 and 0.79 as the exposure condition changes. The value of the exponent may also depend on the duration or time of exposure when the data were collected and the number of data. As more long-term corrosion loss data become available, the existing power models can be modified using numerical optimization procedures to provide a better prediction for the future.

McCuen et al. (1992) provided a numerical optimization procedure to modify the coefficients in the power corrosion model, and it is provided here. Consider the power corrosion model in Equation (2.3). Using a Taylor series expansion, the sum of squares of errors for this model can be approximated as follows:

$$\sum e^2 = \sum [\hat{p} - p]^2 = \sum \left[h_0 \left(\frac{\hat{p}_0 - \hat{p}}{\Delta A} \right) + h_1 \left(\frac{\hat{p}_1 - \hat{p}}{\Delta n} \right) \right]^2 \quad (2.6)$$

where h_0 and h_1 are step sizes for coefficients A and n , respectively; \hat{p}_0 and \hat{p}_1 are the predicted values of p when A and n are incremented by ΔA and Δn , respectively, with the other coefficient held constant; and \hat{p} is the predicted value of p for the estimates of A and n . The iterative procedure can be continued until a minimum value for the *sum of squares of errors* is obtained.

This numerical optimization might result in model coefficients with the best overall goodness-of-fit for the power corrosion model. However, due to the possibly large number of data points at earlier years, the local goodness-of-fit at the earlier years may be better than that at the later years. For an accurate prediction of service life, it is more important to have a model that has better local goodness-of-fit at the later years than at the earlier years. McCuen et al. (1992) suggest a stepwise deletion of earlier data to improve the goodness-of-fit of the power model, especially at the exposure times corresponding to the latest observed value of corrosion loss. Hence, the power corrosion model form can be updated based on the latest field data available.

This literature review could not find any corrosion models that can predict the tension capacity of strands as a function of material, environmental, stress, and void conditions. However, such models are needed to predict the long-term structural capacity of a PT girder and are developed in Section 7. The next subsection provides a review of procedures to determine structural capacity of PT, segmental girders.

2.7. STRUCTURAL CAPACITY OF POST-TENSIONED, SEGMENTAL BRIDGES

2.7.1. Parameters directly influencing structural behavior of PT girders

In Subsection 2.3, a review of material, void, environmental, and stress parameters that directly affect the corrosion and remaining tension capacity of tendons or strands was provided. In this

subsection, a review of parameters that directly influence the flexural capacity of PT girders is provided. These parameters include:

- tension capacity of PT strands, C_T ;
- axial stresses and their losses on strands;
- compressive strength of concrete, f_c ; and
- typical geometrical, structural design, and other parameters of box girders.

2.7.1.1. *Tension capacity of prestressing strands*

In this document, the strands and wires with negligible corrosion are referred to as “as-received” strand and “as-received” wire, respectively. While used in symbols, these terms are abbreviated as “ARS” and “ARW,” respectively. Tension capacities of ARS and ARW are denoted as $C_{T,ARS}$ and $C_{T,ARW}$, respectively.

According to PTI (1998), the minimum ultimate tensile strength (*MUTS*) of a strand is defined as the force equal to the nominal cross-sectional area of a strand, or bar, times its nominal ultimate tensile stress. *MUTS* of a 270 ksi (1860 MPa) grade, 0.6-inch (15.2 mm) diameter, seven-wire “as-received” strand meeting the ASTM A 416 specifications is 58.6 kips (261 kN). This is denoted as $MUTS_{ARS}$, herein. Because of liability concerns, the $C_{T,ARS}$ is generally higher than the $MUTS_{ARS}$. However, this literature review could not find any statistical expressions for $C_{T,ARS}$. Such statistical expressions for “as-received” strands are developed in Subsection 7.3.

As discussed in Subsection 2.3, the C_T can be reduced as a result of corrosion due to the exposure to material, void, environmental, and stress conditions over time. Poston et al. (2003) reported that a 75 percent reduction in strand capacity can result in a 50 percent or more reduction in the live load carrying capacity of the bridge. Moreover, structural failures have been reported due to reduced capacity of strands in voided tendons that are exposed to corrosive environments (NCHRP 1998, ASBI 2000, FDOT 2001a, b, and Hansen 2007). These studies and field observations indicate a critical need to develop time-variant structural reliability models for these bridge structures. This literature review could not find probabilistic expressions for

time-dependent tension capacity of strands exposed to these conditions. However, such expressions are necessary for time-variant reliability assessment of PT bridges and will be developed in this research (refer to Section 7).

2.7.1.2. Axial stresses and their losses on strands

Subsection 2.3.12 discusses the possible effects of axial stresses in the strands, f_{axial} , on corrosion and capacity loss. Two expressions for the effective prestress, f_{pe} , can be expressed as follows:

$$f_{pe} = \begin{cases} f_{pJ} - \Delta f_{pT} \\ or \\ f_{pi} - \Delta f_{pLT} \end{cases} \quad (2.7)$$

where f_{pJ} is the jacking stress; Δf_{pT} is the total prestress losses; f_{pi} is the initial prestress; and Δf_{pLT} is the prestress loss at a given time due to long-term effects. Depending on long-term creep, shrinkage, and other mechanisms, the value of Δf_{pLT} can change as a function of time. The Δf_{pT} can be calculated as follows:

$$f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT} \quad (2.8)$$

where Δf_{pF} , Δf_{pA} , and Δf_{pES} are instantaneous prestress losses due to friction, anchorage seating, and elastic shortening, respectively. The f_{pi} can be calculated as follows:

$$f_{pi} = f_{pJ} - (\Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES}) \quad (2.9)$$

Depending on the available data, an appropriate expression from Equation (2.7) can be chosen to calculate f_{pe} . The value of f_{pe} on individual strands is a key parameter influencing f_{axial} . Because of the variability in the f_{pJ} and Δf_{pT} , different strands may experience different levels of f_{pe} .

Hence, ideally, f_{pe} should be statistically expressed for performing structural reliability analysis. One way to obtain a statistical expression for f_{pe} is to use statistical expressions for the terms in Equation (2.7).

According to TxDOT (2004), the recommended maximum f_{pJ} and average f_{pi} immediately after anchoring are 77 and 70 percent of $MUTS_{ARS}$. These are calculated to be 208 and 189 ksi (1433 and 1304 MPa), respectively. Due to various technical, equipment, measurement, and human error factors, the actual applied f_{pJ} could vary. However, this literature review could not find any statistical expressions for f_{pJ} .

The instantaneous prestress losses (i.e., Δf_{pF} , Δf_{pA} , and Δf_{pES}) in the case of PT, segmental bridges can be minimal because of long tendon lengths and advanced stressing operations and equipment. This literature review could not find sufficient information to develop statistical expressions for Δf_{pF} , Δf_{pA} , and Δf_{pES} . In addition, the variations in Δf_{pLT} also can contribute to variations in the value of f_{pe} for each strand in a segmental girder. It should be noted that for design purposes, f_{pe} is assumed to be the same for all strands. According to the Article C5.9.5.1 in AASHTO (2007), for segmental construction, “calculations for loss of prestress should be made in accordance with a time-step method supported by proven research data....” According to Table 5.9.5.3-1 in AASHTO (2007), the lump sum estimate of Δf_{pLT} in 270-ksi (1861-kMPa) strands in PT box girders is:

$$\Delta f_{pLT} = (19.0 + 4.0PPR) \text{ ksi}; \quad PPR = \text{partial prestress ratio} = \left(\frac{A_s f_y}{A_{ps} f_{py} + A_s f_y} \right) \quad (2.10)$$

where A_s and A_{ps} are cross-sectional areas of conventional and prestressing steels, and f_y and f_{py} are yield strengths of conventional and prestressing steels. For PT, segmental bridges, the PPR can be assumed to be 0. However, according to Article C5.9.5.3 in AASHTO (2007), “For segmental concrete bridges, lump sum losses may be used only for preliminary design purposes....” In addition, this literature review could not find statistical expressions for Δf_{pLT} , especially for PT, segmental bridges.

2.7.1.3. Compressive strength of concrete

The actual and specified compressive strengths of concrete are defined as f_c and f'_c , respectively. Nowak and Collins (2000) compiled information from various research programs and reported key statistical information for f_c (see Table 2-5). From Table 2-5, it seems that the coefficient of variation (COV) decreases as the specified compressive strength, f'_c , increases. Based on this information, a suitable statistical expression for f_c of concrete used in PT bridges in Texas is developed in Subsection 10.5.4.

Table 2-5. Statistics of Concrete Compressive Strength (Nowak and Collins 2000).

Specified compressive strength, f'_c (ksi)	Coefficient of Variation (COV)	Standard deviation (ksi)
3	0.18	0.497
4	0.18	0.610
5	0.15	0.604

1 ksi = 6.89 MPa.

2.7.2. A history of design codes/practices for concrete bridges

AASHTO began publishing the bridge design specifications in the 1930s. The initial documents were based on the Allowable Stress Design (ASD) method that uses only one factor of safety (representing an overall safety factor). In the 1970s, AASHTO made the transition from ASD to the Load Factor Design (LFD) method for bridge design through interim revisions to the existing design documents. Unlike ASD that considers one safety factor, LFD considers different safety factors for each of the load parameters. Bridge designers are making another transition from LFD to Load and Resistance Factor Design (LRFD). The first five editions of *The AASHTO Specifications for Design of Highway Bridges* (AASHTO 1994, 1998, 2004, 2006, 2007) incorporate limit state design philosophies that are based on structural reliability methods to achieve a more uniform safety level in new bridge designs. Also, in 2000, AASHTO and FHWA set October 1, 2007, as the transition date for all the government-initiated bridges to be designed

following the AASHTO LRFD Specifications. The AASHTO LRFD Specifications are general design documents for all types of bridges.

Due to special construction sequences and other reasons, segmental concrete bridges need special design considerations. To meet this requirement, AASHTO published *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges* (AASHTO 1999), which is widely used in the segmental bridge industry. The AASHTO (1999) was developed based on the findings from the extensive research conducted at the Center for Transportation Research, at University of Texas, Austin. Until now, no modifications have been made to AASHTO (1999), and many sections in AASHTO (1999) refer to AASHTO (1998).

2.7.3. Differences in the behavior of monolithic and segmental, post-tensioned beams

Section 4.12.5.4 in Nawy (2003) provides a scientific explanation supporting the suitability of using the flexural formulations for sections with bonded tendons for analyzing the sections with unbonded tendons. However, this explanation seems to be provided for unbonded tendons in monolithic beams and not for segmental beams with external tendons. The key idea behind this explanation is the fact that the non-prestressed reinforcement undergoes larger strain than its yield strain near failure situations (i.e., post-elastic range) and controls the flexural crack development and width. In a segmental bridge span with several segments, the non-prestressed reinforcement is generally not extended across the transverse joints between the segments. Hence, the above-mentioned explanation for monolithic beams does not suit the case of segmental beams.

2.7.4. Stress-strain relationship for unbonded tendons

According to Naaman and Alkhairi (1992), AASHTO (1996), and Tabsh (1995), for PT beams with unbonded, external tendons, the average axial stress at nominal or ultimate conditions, f_{ps} , on tendons is influenced by the load configuration and span, in addition to the material and longitudinal and cross-sectional properties. They also found that f_{ps} of the unbonded tendons is always less than its yield strength, f_{py} , which is the limiting value of f_{ps} in AASHTO (1996). Sections 5.7.3.1.1 and 5.7.3.1.2 in AASHTO (2007) simplified these equations and provided

separate formulations to determine the average stress, f_{ps} , in bonded and unbonded tendons, respectively. The formulation in 5.7.3.1.2 of AASHTO (2007) can be used to determine the f_{ps} in tendons in order to assess the flexural capacity of segmental bridges with unbonded tendons (both internal and external).

In the case of PT bridges with external tendons, the flexural capacity assessment is further complicated when the strands have different distances from the extreme top fiber, different effective prestress levels, and different cross-sectional areas due to different corrosion rates and resulting changes in tension capacity. These corrosion rates can be different because of varying material, environmental, void, and stress factors. A procedure to determine the probabilistic moment capacity of PT girders with external, unbonded tendons needs to be developed. The next subsection provides a review of parameters that directly affect the structural capacity of PT girders.

2.7.5. Stress-strain relationship for concrete cross-sections

To assess the structural behavior under normal service loading conditions, the stress-strain relationship in concrete can be assumed to be linear. This will provide a sufficient level of accuracy, if deformations are minimal. However, as the loading conditions become severe, especially at ultimate behavior, the stress-strain relationship in concrete becomes non-linear. Design documents such as AASHTO (2007) use the Whitney stress block, which is a close approximation of this non-linear behavior. More accurate non-linear stress-strain relationships have been developed by Hognestad (1951) and Todeschini et al. (1964) are more appropriate for assessing behavior at ultimate conditions. The models by Hognestad (1951) and Mander (1988a, b) divide the concrete cross-section into several layers and then determine stresses in each layer. On the other hand, Todeschini et al. (1964) provide a closed-form solution for determining stress in concrete. Considering the time required to perform numerous simulations required in structural reliability analysis, the model developed by Todeschini et al. (1964) provides computational efficiency.

2.7.6. In-service and allowable stresses in a cross-section

The applied compressive and tensile stresses at service at extreme fibers can be calculated using the following expressions provided in the PCI (1999) handbook and other text books, including Nawy (2003) and Naaman (2004):

$$\begin{aligned} \text{Compressive stress at extreme top fiber, } f_t &= \left[-\frac{P_e}{A_c} \left(1 - \frac{ec_t}{r^2} \right) - \frac{M_T}{S_t} \right] \\ \text{Tensile stress at extreme bottom fiber, } f_b &= \left[-\frac{P_e}{A_c} \left(1 + \frac{ec_b}{r^2} \right) + \frac{M_T}{S_b} \right] \end{aligned} \quad (2.11)$$

where M_T is the total applied moment; P_e is the effective prestress force after losses; e is the eccentricity of each strand from the center of gravity of the concrete cross-section; r is the radius of gyration; A_c is the area of the concrete cross-section; S_t and S_b are section moduli of the concrete section about the extreme top and bottom fibers, respectively; and c_t and c_b are the distance of the extreme top and bottom fibers, respectively, from the centroid of the concrete cross-section.

Table 5.9.4.2.1-1 and Table 5.9.4.2.2-1 in AASHTO (2007) recommends compressive and tensile stress limits, respectively, in fully prestressed components in segmental concrete at the service limit state after prestress losses. These are reproduced in Table 2-6.

Table 2-6. Compressive and Tensile Stress Limits for Fully Prestressed Components in Segmental Concrete at Service Limit State after Losses (AASHTO 2007).

Stress type		Stress limit
Longitudinal compressive stresses in extreme concrete fibers	Due to the sum of effective prestress and permanent loads	$0.45 f'_c$
	Due to the sum of effective prestress, permanent loads, and transient loads and during shipping and handling	$0.60 \phi_w f'_c$
Longitudinal tensile stresses through joints in the precompressed tensile zone	At joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of $0.5 f_y$; internal or external tendons	$0.0948 \sqrt{f'_c}$ (in ksi) or $3 \sqrt{f'_c}$ (in psi)
	At joints without the minimum bonded auxiliary reinforcement through joints	No tension

In Table 2-6, one of the compressive stress limits uses the stress limit reduction factor, ϕ_w . Per Article 5.9.4.2 in AASHTO (2007), the use of ϕ_w to reduce the stress limit in the walls of box girders at service limit state is not theoretically correct, but is a rational approach. The procedures to calculate ϕ_w are provided in Article 5.7.4.7.1 in AASHTO (2007).

2.7.7. Existing approaches to determine flexural capacity of prestressed girders

Section 5.7 of AASHTO (2007) provides empirical and analytical formulations for flexural design of conventionally reinforced and prestressed concrete beams. However, difficulties exist in directly using these formulations for flexural analysis of a girder with corroding strands. Computer programs need to be developed to accommodate for the effect of changes in effective prestress, prestress losses, and tension capacity of individual strands on flexural capacity of the girder. Ting and Nowak (1991) reported that unlike the case of a reinforced concrete beam, the formulation of an analytical model for flexural behavior of a prestressed beam is complex because of the presence of prestressing strands instead of conventional reinforcement.

Ting and Nowak (1991) provided an iterative algorithm to determine the curvature and depth of the neutral axis for a particular bending moment in a rectangular pre-tensioned beam with a composite reinforced concrete slab. This algorithm is based on the strain compatibility approach and is applicable for pre-tensioned beams with internal, bonded tendons, but not for PT

beams with external, unbonded tendons. This is because, according to AASHTO (1999), the strain compatibility approach can be used for unbonded tendon systems only if “the analysis correctly recognizes the differences between the tendons and the concrete section, and the effect of deflection geometry changes on the effective stress in the tendons...” In addition, the model developed by Ting and Nowak (1991) needs modifications to account for the varying capacity of strands located at the same depth (say, different strands within a tendon, especially when strand corrosion occurs).

Cavell and Waldron (2001) developed a non-linear flexural capacity model for PT beams with internal, grouted, and bonded tendons. This model was based on the strain compatibility approach with modifications to account for the effect of locally unbonded tendon regions at void locations and the flexural capacity loss due to failed tendons. However, this model cannot be used for beams with external tendons because of the difference in the average stress on external, unbonded tendons when compared to that in internal, bonded tendons. These analytical approaches with necessary modifications can be used to determine the flexural capacity of segmental bridges with external, unbonded tendons.

2.8. STRUCTURAL DEMAND ON POST-TENSIONED BRIDGES

The estimation of structural reliability may be performed on the basis of the unfactored, unbiased demand loads. These demand loads may be obtained using basic structural analysis principles (such as the influence line theorem), suitable statistical expressions, and recommendations from AASHTO (2007) for dead, live, and impact loads.

2.8.1. Dead load

In general, the dead load, DL , can be obtained by adding the weight of the steel, concrete, and overlay materials on the girder elements. Table 2-7 provides information that can be used to determine the average dead load on a bridge. It is usual to consider DL as a constant parameter for bridges throughout their service life. However, to account for designers' tendency to underestimate the dead load, Table 6.1 in Nowak and Collins (2000) suggests expressing dead load parameters in terms of a normal distribution with bias factors (mean-to-nominal ratio) and

COVs of 1.05 and 0.10, respectively. However, because the dead load cannot usually be negative, a lognormal distribution is assumed for dead load parameters in Section 10.

Table 2-7. Dead Load Parameters from TxDOT Bridge Drawings.

Factors contributing towards the dead load	Unit weight
Reinforced concrete	155 pcf (0.025 N/m ³)
Wearing surface (concrete overlay)	25 psf (0.0012 MPa)
Future wearing surface (concrete overlay)	25 psf (0.0012 MPa)
Concrete barrier rails (on each side of the roadway)	326 plf (4757 N/m) per rail

2.8.2. Live and impact loads

Live and impact loads are induced by moving vehicles and their vertical dynamic actions. Live loads can be determined by considering various factors, including the clear span, total vehicular weight, axle loads and configurations, transverse and longitudinal position of vehicles, and structural stiffness. The HS20 live load model was used to design highway bridges until the AASHTO (1996) document began to be used for design purposes. According to TxDOT (2004), PT bridges in Texas were designed following the AASHTO (1982) Standard Specification to carry the dead load and live load with three lanes of AASHTO HS20-44 loading with allowance for impact and alternate military loading.

The AASHTO (1998) document suggests a heavier live load model. This live load model is known as HL-93 loading (Figure 2-16) and is also recommended in Section 3.6 of AASHTO (2007). Figure 2-16 is an approximately scaled drawing.

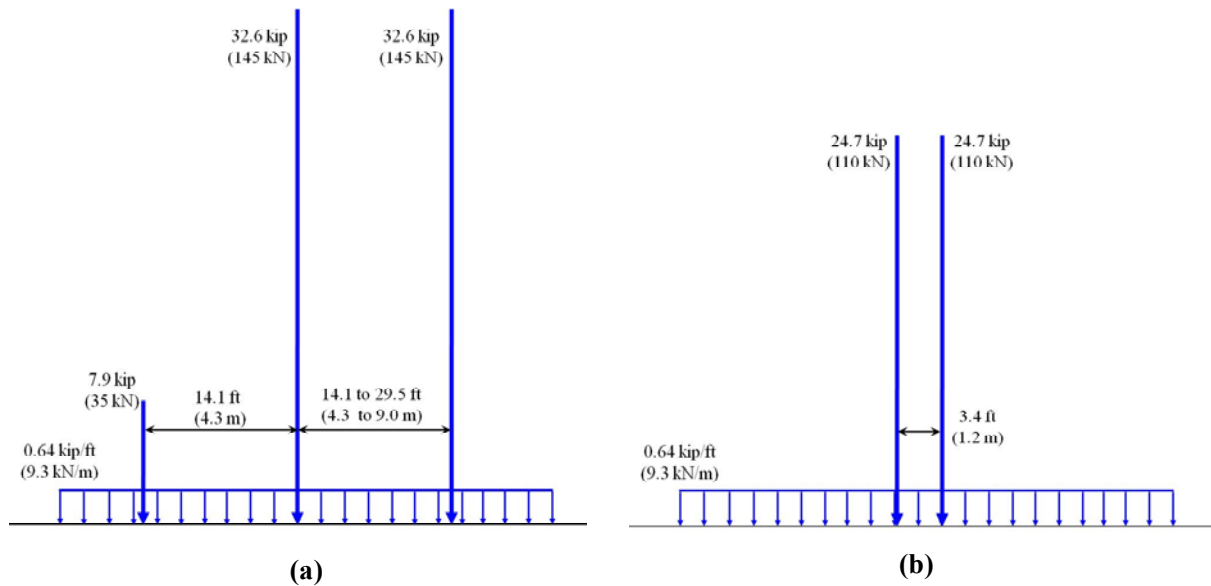


Figure 2-16. HL-93 Loading Specified by AASHTO (1998, 2007): (a) Truck and Uniform Lane Load and (b) Tandem and Uniform Lane Load.

The HL-93 loading is a combination of a design truck or design tandem and a design lane load. Nowak and Collins (2000) reported a bias factor of 1.25 for live load. In addition, the impact loading is considered equivalent to 33 percent of the live load (AASHTO 2007). According to Nowak and Collins (2000), the COV of the joint effect of live and impact loading is 0.18. A normal distribution based on this information can be assumed for live load distribution. AASHTO (2007) also provides a multiple-lane factor to calculate the total live load. The multiple-lane factors are 1.20, 1.00, 0.85, and 0.65 for one, two, three, and greater than three lanes, respectively. Further details on the procedures to determine total demand loads are provided in the AASHTO (2007) document and are not reproduced here. Based on the random realization of the load parameters and their configuration, the moment demand, D_M , can be computed using standard structural analysis methods.

2.9. STRUCTURAL RELIABILITY OF POST-TENSIONED BRIDGES

This literature review could not find significant information on the structural reliability of PT bridges, especially those with voided tendons. This section reviews some of the existing

work on structural reliability of bridges and existing methodologies that could be useful for developing a structural reliability model for PT bridges.

2.9.1. Structural reliability of bridges with uncorroded strands

In the past, the strength and service reliabilities have been generally assessed based on flexural capacity of and allowable stresses on the structural element, respectively. Tabsh (1995) studied flexural reliability of cast-in-place PT slab bridges with unbonded tendons. This study found that the AASHTO *Standard Specifications for Highway Bridges* (AASHTO 1992) underestimates the stresses in unbonded tendons at ultimate flexural conditions, especially if the reinforcement index (defined as the ratio of the area of steel reinforcement to total area of cross-section) is low. The amount of reinforcement influences the statistics of the axial stress experienced by unbonded tendons and the bending stress experienced by the beam. These observations indicate the importance of the amount of steel reinforcement, which could decrease due to time-dependent corrosion, on structural capacity. However, Tabsh (1995) did not consider the possible capacity loss of strands and found that, based on the “as-received” strand condition, the β for typical AASHTO slab bridges can vary between 2.75 and 5.0. The values of β can decrease as the available cross-sectional area (or tension capacity) of strands decrease. Hence, this emphasizes the need for probabilistic models for C_T of corroding strands in order to estimate structural capacity and reliability with better accuracy and precision.

Du and Au (2005) performed a comparative study on the structural reliability of a pre-tensioned T-girder assessed based on AASHTO (1998), Chinese code (CDCHB 1991), and Hong Kong code (SDMHR 1997). Du and Au (2005) determined the governing limit states for prestressed concrete girders with long and short spans designed using these codes, and these are shown in Table 2-8.

Table 2-8. Governing Limit States for Pre-tensioned Girders with Short and Long Spans.

Design Code	Short span (i.e., 25 to 30m [82 to 98 ft])	Long span (i.e., 35 to 40m [115 to 131 ft])
AASHTO LRFD	Service limit state	Service limit state
Chinese	Service limit state	Service limit state
Hong Kong	Strength limit state	Service limit state

Based on the fact that the service limit state governs most cases, Du and Au (2005) recommended that the bridge design codes be calibrated based not only on the strength limit state but also on the service limit state.

Because the structural reliability varies as a function of time, methodologies have to be developed to assess time-variant structural reliability based on strength and service limit state conditions. A review of available techniques to model and assess structural reliability is provided next.

2.9.2. Modeling and assessment of structural reliability

2.9.2.1. Limit state functions

The definition and determination of an appropriate limit state function (or a performance function) is essential in modeling structural reliability. Following the conventional notation in structural reliability theory (Ditlevsen and Madsen 1996), the limit state function, g , can be formulated as:

$$g(C, D) = C - D \quad (2.12)$$

where C and D represent capacity and demand, respectively, such that the events with $g \leq 0$ represent failure. In general, the limit state functions for strength and service criteria are used to assess structural reliability. Modeling and evaluating appropriate limit state functions for complex structural systems such as PT, segmental bridges pose challenges, and these could not

be found in this literature review. The current research addresses these challenges through modeling strength and service limit states for segmental bridges (see Section 10).

2.9.2.2. *Generalized structural reliability index and probability of failure*

The structural reliability can be expressed in terms of the generalized reliability index, β , defined as follows (Ditlevsen and Madsen 1996):

$$\beta = -\Phi^{-1}(P_f) = -\Phi^{-1}(P(g \leq 0)) \quad (2.13)$$

where Φ is the cumulative distribution function of the standard normal distribution and P_f is the probability of failure (estimated as the probability of $g \leq 0$). Standard procedures such as the closed-form solution, first order reliability method (FORM), and second order reliability method (SORM) can be used to determine β , when g is simple and differentiable. However, these methods are not suitable when g is complex and non-differentiable, as in the case of PT bridges. In such cases, Monte Carlo simulation techniques can be used to determine β . In Monte Carlo simulation techniques, the probability of failure, P_f , is determined as follows:

$$P_f = \frac{N_f}{N_{sim}} \quad (2.14)$$

where N_f indicates number of failure instances and N_{sim} indicates the number of simulations of g . The estimated P_f can be used to determine β using Equation (2.13). Accuracy of P_f is reflected in the accuracy of β . A sufficient level of accuracy in the estimation of P_f can be attained by setting an upper limit on the coefficient of variation of P_f , $COV(P_f)$. $COV(P_f)$ can be determined as follows (Ditlevsen and Madsen 1996):

$$COV(P_f) \approx \frac{1}{P_f} \cdot \sqrt{\frac{(1-P_f)P_f}{N_{sim}}} \quad (2.15)$$

Various computer programs are available to use as a platform to perform these calculations and then to estimate β . These computer programs are discussed next.

2.9.2.3. *Assessing structural reliability using existing computer programs*

Der Kiureghan et al. (2006) provided discussions on three reliability related software programs developed at the Pacific Earthquake Engineering Research Center (PEER) at the University of California, Berkeley. The three major software programs developed by researchers at PEER were CalREL, *Open System for Earthquake Engineering Simulation* (OpenSEES), and *Finite Element Reliability Using MATLAB* (FERUM).

CalREL was initially developed by Liu et al. (1989) and was modified later by researchers at PEER. CalREL is a general-purpose structural reliability analysis program written in FORTRAN[®]. CalREL incorporates FORM, SORM, importance sampling, and Monte Carlo simulation techniques for reliability analysis. CalREL provided the technical basis for the development of FERUM and OpenSEES (Der Kiureghan et al. 2006).

OpenSEES (Der Kiureghan et al. 2006), developed at PEER, is an object-oriented, general purpose program written in C⁺⁺ language and was developed for earthquake-engineering applications. OpenSEES is a platform that can combine the static and dynamic non-linear finite element analysis and reliability methods. The capabilities of OpenSEES include reliability analysis by FORM, SORM, importance sampling, and Monte Carlo simulation.

FERUM is a MATLAB[®] toolbox consisting of a set of MATLAB[®] functions. FERUM can be used to perform reliability analysis of limit state functions that are defined using either finite element or other analytical methods. Version 3.1 of FERUM can perform reliability analyses using approaches such as FORM, SORM, importance sampling simulation, system reliability, and inverse FORM. According to Der Kiureghan et al. (2006), the most attractive features of FERUM are: 1) the ease in modifying the limit state function(s), 2) user-friendly debugging and error handling utilities, and 3) the ability to use numerous mathematical functions available with MATLAB[®].

Traditionally, these programs have been mainly used in the reliability assessment of structural systems exposed to various earthquake scenarios. However, these computer programs can also be used for assessing time-variant reliability of deteriorating structural systems, provided g is probabilistically formulated. In the case of this research on the structural reliability of segmental bridges, FERUM was determined to be a useful tool to incorporate g and then assess β . The next step is to compare the available β with the required level of reliability.

2.9.3. Target reliability index

Various parameters such as β and condition rating factor exist to assess structural safety. According to Akgul and Frangopol (2004), β is a measure of the structural reliability with respect to the limit state functions, whereas the condition rating factor is a measure of the reserve structural capacity with respect to the applied load. The latter can be a qualitative or deterministic assessment parameter, while the former is a fully or semi-probabilistic assessment parameter. Because, in reality, the structural behavior and loading conditions can be truly probabilistic in nature, β is preferred over the condition rating factor. Structural design is now being performed using standardized design codes that are calibrated such that a minimum required β is achieved during the design/construction phase. This β during the design/construction stage can be defined here as the initial reliability index, β_0 . For structures with no deteriorated structural elements, the desired value of β (based on flexural capacity) is generally higher than 3.5 (indicating a corresponding P_f less than 2.33×10^{-4}). Tabsh (1995) found that the value of β for typical AASHTO cast-in-place PT slab bridges with uncorroded strands can vary between 2.75 and 5.0 (corresponding values of P_f are 3.0×10^{-3} and 2.87×10^{-7} , respectively) However, the β of a structure can decrease over time as materials deteriorate (e.g., strand corrosion in the case of PT bridges) over time.

To ensure sufficient levels of safety during service, β of existing structures must be greater than a minimum required value. This minimum required value of β is defined as the target reliability index, β_{target} . Normally, the design specifications such as AASHTO (2007) are calibrated based on a β_{target} equal to 3.5 for flexural analysis of structural systems (Nowak and Collins 2000). According to ISO 13822, the fundamental differences in various considerations

during design/construction and assessment/maintenance/repair procedures for a structure can be categorized into three distinct groups as follows:

- *Economic considerations* – issues related to the cost to increase the safety of an existing structure. This cost can be higher than that required to increase the safety level of a structure during the design phase.
- *Social considerations* – issues related to the disruption and displacement of current users and their activities as an effect of post-assessment repair works. This is generally not an issue during the design/construction phase.
- *Sustainability considerations* – issues related to the disposal, recycling, or both of structural components during or after the repairs. This is generally not an issue during the design/construction phase.

Values of β_{target} used for different g functions in structural reliability assessment procedures, as provided in ISO 13822, are summarized in Table 2-9. The corresponding P_f values are also shown in Table 2-9.

Table 2-9. Summary of β_{target} and P_f (Adapted from ISO 13822).

Reference Period	Limit States		β_{target}	$P_f = \Phi(-\beta_{\text{target}})$
Remaining service life	Serviceability	CoF – Reversible	0.0	5.0×10^{-1}
		CoF – Irreversible	1.5	6.7×10^{-2}
	Fatigue	Inspection is possible	2.3	1.1×10^{-2}
		Inspection is not possible	3.1	9.7×10^{-4}
Design life	Ultimate strength (e.g., flexural)	CoF – Very Low	2.3	1.1×10^{-2}
		CoF – Low	3.1	9.7×10^{-4}
		CoF – Medium	3.8	7.2×10^{-5}
		CoF – High	4.3	8.5×10^{-6}

CoF indicates “Consequences of Failure.”

In Table 2-9, note that for each type of limit state, the higher the consequences of failure (CoF), the higher the value of β_{target} will be. For example, a nuclear reactor may have a larger CoF than an urban highway bridge, which in turn might have a larger CoF than a rural highway bridge. Accordingly, β_{target} for a nuclear reactor will be higher than that for an urban highway bridge, and β_{target} for an urban highway bridge will be higher than that for a rural highway bridge.

Also, note that β_{target} for the ultimate strength (e.g., flexural capacity) limit state is greater than that for the service (i.e., allowable stress) limit state. To assess the structural reliability based on strength and service limit states, typical parameters of segmental bridges need to be characterized. These characteristics are presented next.

2.10. TYPICAL CHARACTERISTICS OF SEGMENTAL CONCRETE BRIDGES IN TEXAS

2.10.1. Segmental concrete bridges in Texas

Depending on structural, aesthetic, and other requirements, the geometrical and structural design parameters of segmental box girders vary from one bridge to another and even within the same bridge. There are 10 major segmental bridges (existing or under construction/design phases) in Texas (see Figure 2-17). The existing bridges are located in San Antonio, Austin, Corpus Christi, Port Arthur, Matagorda Island, and Dallas. The bridge at the Interstate Highway-10 (I-10) crossings over the Trinity and Neches rivers are under design/construction phase now. The intra-coastal bridge in Corpus Christi was constructed in the 1970s and was the first precast segmental bridge in the US. This bridge has only internal tendons grouted with Class B grout with an expansive admixture. The segmental bridges in Austin, Port Arthur, and San Antonio were constructed in the 1980s and have both internal and external tendons grouted with Class B grout with an expansive admixture. Other bridges shown in Figure 2-17 were constructed in the 2000s or are under construction and have both internal and external tendons with high performance Class C grout.

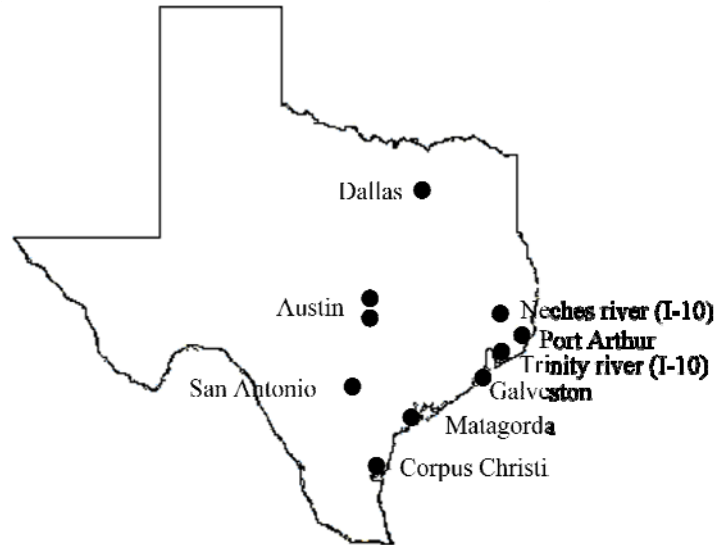


Figure 2-17. Locations of Existing and Future Post-Tensioned, Segmental Concrete Bridges in Texas.

2.10.2. San Antonio “Y” bridge

In the late 1980s and early 1990s, the San Antonio “Y” bridge, an urban viaduct, was designed using AASHTO (1982) and constructed using span-by-span techniques. Usually, in this technique, multiple segments within a span are first positioned in place and externally supported using rigid steel frames. The longitudinal tendons travelling between the outer ends of the exterior segments (placed on the piers) are then post-tensioned. The rigid supports are released after post-tensioning all the tendons. The bridges constructed this way behave like a simply supported beam. In most spans in the San Antonio “Y” bridge, some tendons are post-tensioned before and some tendons are post-tensioned after releasing the rigid steel supports. Because of this special post-tensioning sequence, most spans in the San Antonio “Y” bridge do not behave like simply supported beams. In addition, some tendons travel across multiple spans, indicating a structural behavior similar to that of a continuous-support beam. However, this is also not true because some tendons are post-tensioned before and some tendons are post-tensioned after releasing the rigid steel supports. In summary, the San Antonio “Y” bridge behaves neither like a simply supported nor like a continuous-support beam. A layout of the San Antonio “Y” bridge and its different phases is provided in Figure 2-18.

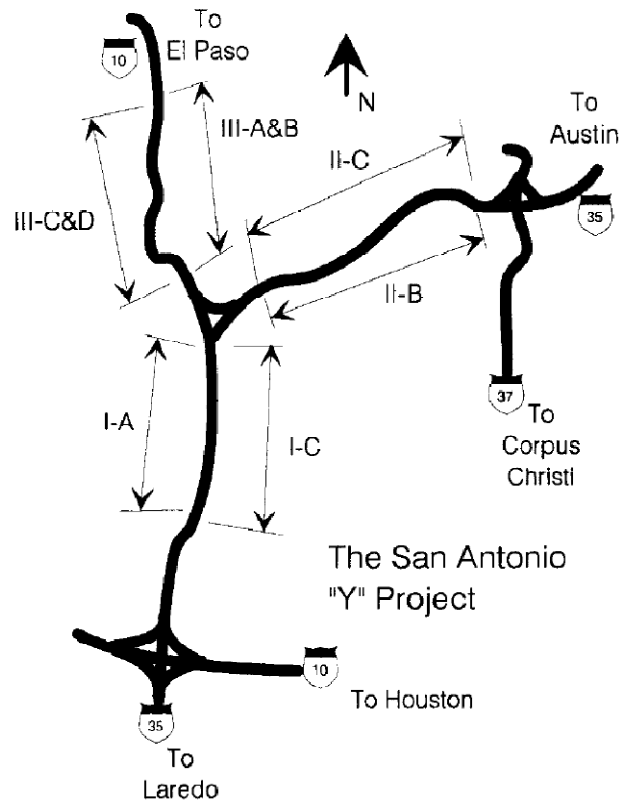


Figure 2-18. A Layout and Phases of San Antonio “Y” Bridge (Wollmann et al. 2001).

The San Antonio “Y” bridge is located at the intersection of I-10 and I-35 highways around downtown San Antonio, Texas. McGray (1998) determined the traffic density, especially due to the US-Mexico trade, on US highways. Graphical representations of these data are provided by McGray (1998) and are reproduced in Figure 2-19 and Figure 2-20. It is clear that the San Antonio “Y” bridge has a high importance measure compared to most other segmental bridges in Texas and other states.

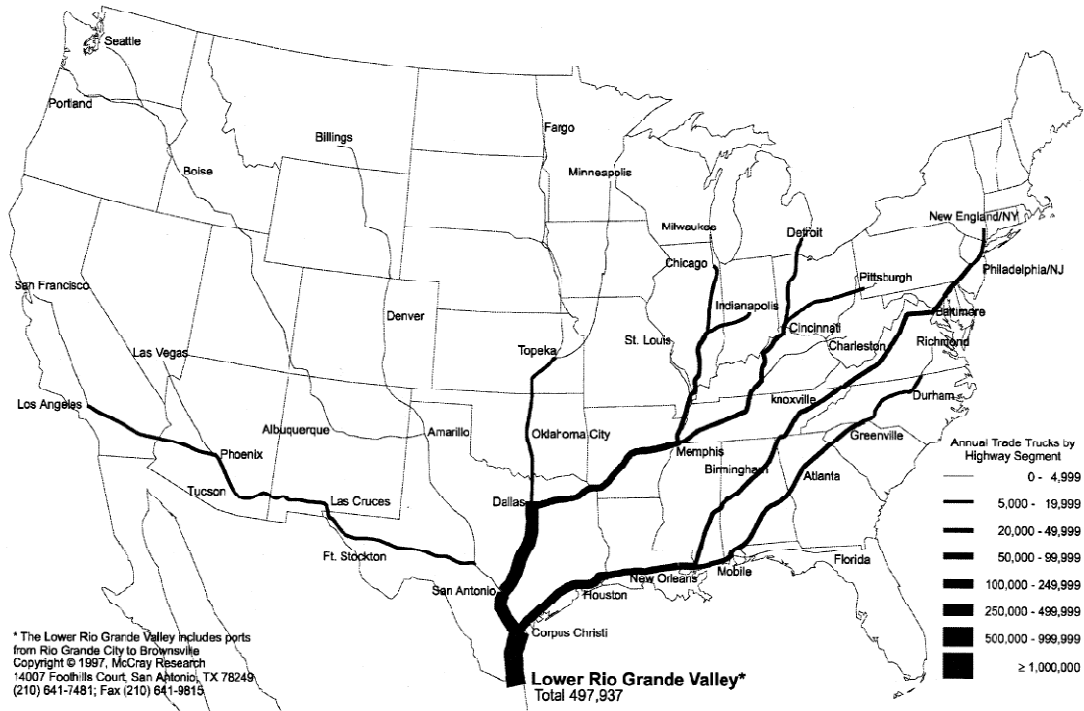


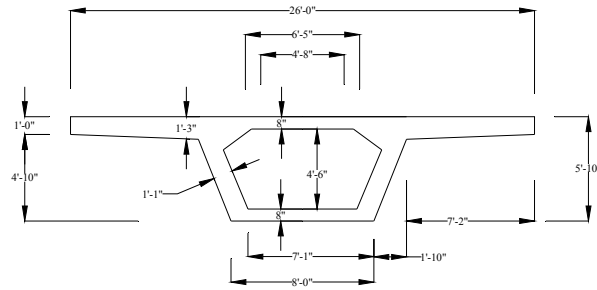
Figure 2-19. Estimated Trucks Carrying US-Mexico Trade on US Highway Corridors to and from the Lower Rio-Grande Valley Region (McGray 1998).



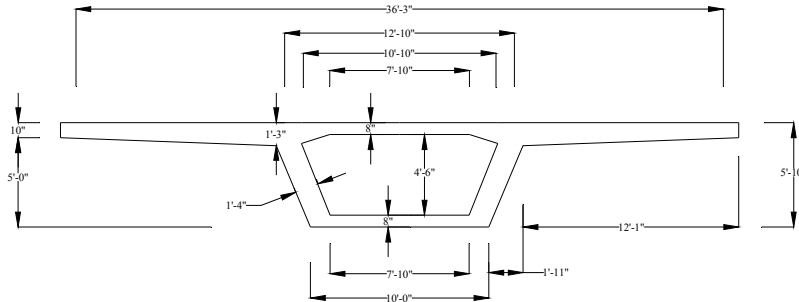
Figure 2-20. Estimated Trucks Carrying US-Mexico Trade on US Highway Corridors to and from the Laredo Region (McGray 1998).

2.10.3. Span and girder inventory in San Antonio “Y” bridge

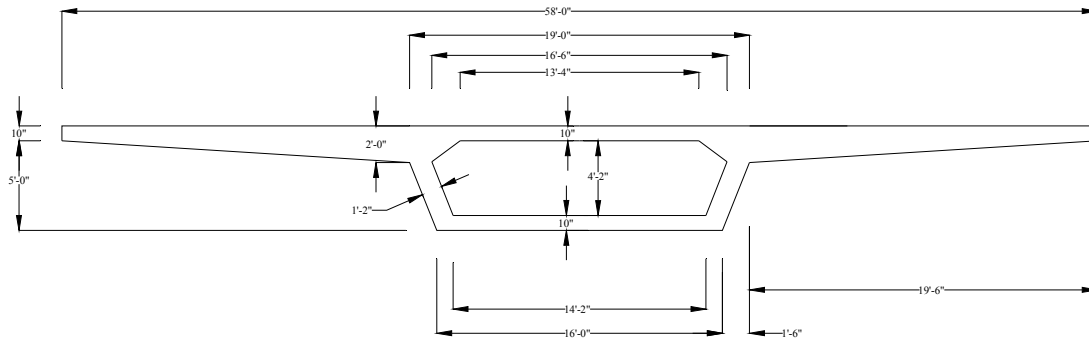
This bridge system is divided into various spines and ramps. These spines and ramps are constructed using precast concrete segments with different basic cross-sectional properties (Figure 2-21). Table 2-10 provides span and girder inventory on the bridge.



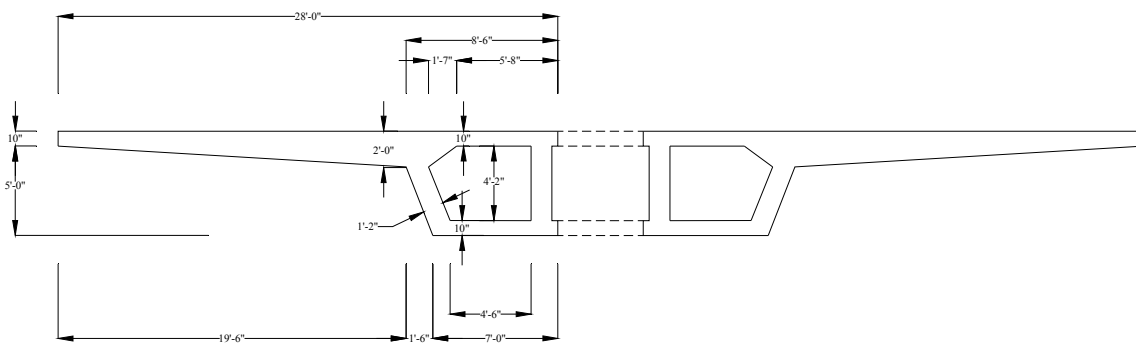
Type I Superstructure Segment (San Antonio "Y"; TxDOT Bridge Drawings, Page 1225)



Type II Superstructure Segment (San Antonio "Y"; TxDOT Bridge Drawings, Page 1227)



Type III Superstructure segment (San Antonio "Y"; TxDOT Bridge Drawings, Page 1229)



Type IV Superstructure segment (San Antonio "Y"; TxDOT Bridge Drawings, Page 1231)

Figure 2-21. Typical Cross-Sections of Four Types of Girders in San Antonio “Y” Bridge.

Table 2-10. Span and Girder Inventory of San Antonio “Y” Bridge.

Bridge Name	Type of girder	No. of traffic lanes	Flange width (ft)	Number of spans	Span (ft)	Overall length (ft)	Overall traffic lane length (ft)
IH-10	II	3	58	79	100	8030	24,090
	I	1	26	50	100	5129	5129
	I	3	58	89	100	8955	26,866
	II	1	26	9	100	913	914
	II	1	26	11	100	1157	1157
	I	3	58	11	100	1111	3335
	I	1	26	8	100	810	810
IH-35 Southbound	I / III	2	42/48	78	100	7448	14,896
	I	2	42	51	100	5000	10,000
	I	2	42	51	100	5028	10,056
	I / III	2	42/56	76	100	7468	14,937
	I	2	42	4	100	401	802
	II	1	26	2	100	190	190
	II	1	26	5	100	481	481
	I	1	42	7	100	720	720
	II	1	26	8	90	740	740
	I / II	2	42/26	12	90	1079	2,158
	II	1	26	3	100	284	284
	II	1	26	6	100	578	578
IH-35 Northbound	III / IV	3	58/58	60	100	6103	18,310
	II/III/IV	3	26/58/67	63	90	5701	17,104
	I	1	26	7	100	794	794
	II	1	26	15	90	1384	1,384
	II	2	38	18	100	1712	3,423
	II	2	38	18	100	1672	3,344

Note: 1 ft = 0.3048 m

The cumulative overall length of Type I, II, III, and IV girders are 58,512, 55,847, 65,248, and 35,414 ft (17,834, 17,022, 19,888, and 10,794 m), respectively. This indicates widespread use of Type I, II, and III girders. It was also observed that Type I girder with relatively narrow flange widths are mainly used for ramps (with 26,866 ft (8,189 m) of these on Ramp E). Also, Type II, III, and IV girders with relatively wide flanges are used mainly on spines. It is assumed here that the spines, which constitute main traffic lanes, have higher significance or importance factors than ramps. Due to these reasons, Type II and III girders can be pre-selected for structural performance and reliability studies.

2.10.4. Tendon inventory on San Antonio “Y” bridge

The San Antonio “Y” bridge consists of both internal and external PT systems. Table 2-11 provides the total number of internal and external longitudinal strands in the different girder types used in Phase II-C of the bridge system, as shown in Figure 2-18. Also, Table 2-11 shows that these girders have different longitudinal tendon layout and different numbers of span inventory. It also provides information on the depth of the centroid of the tendon from the extreme concrete compression fiber. Among the Type II and III girders, Type III girders are more widely used and, hence, will be assumed to be a typical girder cross-section for the reliability studies.

Table 2-11. Quantity and Location of Longitudinal Tendons at Midspan on San Antonio “Y” Bridge.

Girder type	No. of spans	Total number of strands in tendons			
		External tendons		Internal tendons	
		Depth of centroid from extreme top fiber (inches [mm])			
		54 (1.37)	54 (1.37)	40 (1.02)	66 (1.68)
Type I	3	2 x 7	-	6 x 12	-
	1	4 x 12	-	6 x 12	-
	3	2 x 12	-	6 x 12	-
	2	4 x 19	-	6 x 12	-
	3	2 x 19	-	6 x 12	-
	2	-	-	6 x 12	-
Type II	3	1 x 7	-	6 x 12	-
	1	1 x 12	-	6 x 12	-
	6	2 x 7	-	6 x 12	-
	1	4 x 12	-	6 x 12	-
	9	2 x 12	-	6 x 12	-
	12	3 x 19	-	6 x 12	-
	3	2 x 19	-	6 x 12	-
Type III	1	1 x 7	-	6 x 12	-
	5	4 x 12	-	4 x 19	4 x 12
	4	4 x 12	-	4 x 19	4 x 12
	4	2 x 12	-	4 x 19	4 x 12
	17	6 x 19	-	4 x 19	4 x 12
	17	4 x 19	-	4 x 19	4 x 12
Type IV	3	2 x 19	-	4 x 19	4 x 12
	1	2 x 7	-	4 x 19	4 x 12
	4	2 x 12	-	4 x 19	4 x 12
	2	2 x 19	4 x 12	4 x 19	4 x 12
	3	2 x 19	2 x 12	4 x 19	4 x 12
	1	2 x 19	-	4 x 19	4 x 12
1	2 x 19	-	4 x 19	4 x 12	

The number before and after the “x” sign indicates number of tendons and number of strands in each tendon.
1 inch = 25.4 mm

2.10.5. Cross-sectional properties of girders in San Antonio “Y” bridge

A schematic with symbols for different geometrical parameters of the cross-section of segmental box girders is shown in Figure 2-22. Table 2-12 provides typical values for these geometrical parameters. Earlier, in Figure 2-21, the cross-sectional views of four typical box sections in the

San Antonio “Y” bridge were provided. It could be easily identified that Type I has less flange width in comparison to the other three girder types. This is consistent with general bridge dimensions where ramps have less dimensions compared to main lanes or spines.

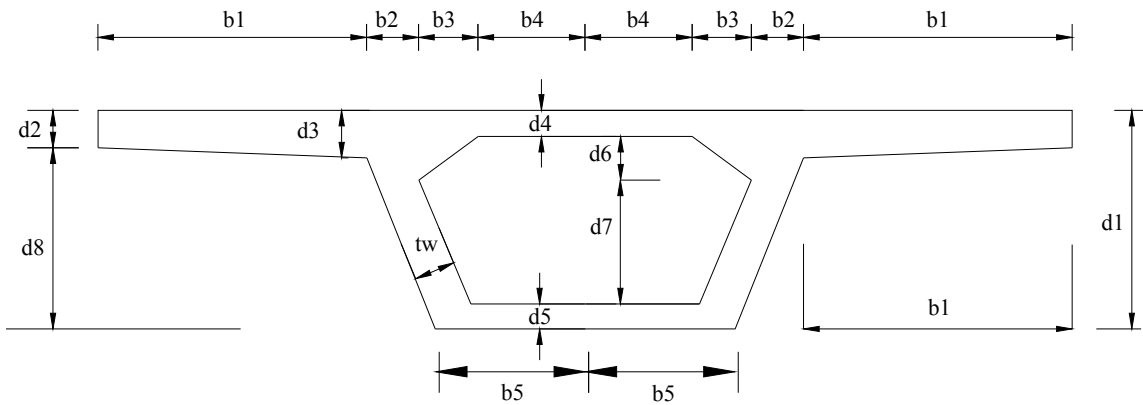


Figure 2-22. Symbolic Notations for Cross-Sections of Segmental Box Girders.

Table 2-12. Typical Geometrical Parameters of Box Girders in San Antonio “Y” Bridge.

Parameter (Symbol)	Type I	Type II	Type III	Type IV
b1	74	136	222	-
b2	16	14	15	-
b3	19	19	19	-
b4	35	47	80	-
b5	48	60	96	-
t_w	12	12	14	14
d1	70	70	70	70
d2	10	10	10	10
d3	15	19	25	25
d4	8	8	10	10
d5	8	8	10	10
d6	14	14	14	14
d7	40	40	36	36
d8	55	51	45	45

Note: 1 inch = 25.4 mm

The structural reliability studies presented in Section 10 use a cross-section similar to that of the Type III girder used on the San Antonio “Y” bridge but may not be exact. However, other assumptions used for the reliability analysis may not represent the structural and exposure conditions experienced by the San Antonio “Y” bridge, as the objective of this research is only to develop models for typical PT bridges. Hence, the structural reliability analysis presented in Section 10 of this document does not intend to represent that of the San Antonio “Y” bridge.

2.11. SUMMARY

A comprehensive review of literature on failure cases of PT bridges, mechanisms and parameters influencing corrosion and tension capacity of strands/tendons, existing corrosion models for immersion and atmospheric exposure conditions, parameters influencing structural capacity of PT girders, and procedures for determining structural capacity and reliability of PT girders were performed. Then a review of typical characteristics of PT bridge girders in Texas was reviewed. This information will be used to meet the goals and objectives of this research program. However, note that the combination of PT bridge characteristics and support and exposure conditions used in the structural reliability studies and the results from this study presented in Section 10 do not represent any specific bridge in Texas. The following section provides a brief discussion on current research needs and significance of this research.

3. CURRENT NEEDS AND RESEARCH SIGNIFICANCE

Equation Section (Next)

Bridge inspections have reported the presence of voids, moisture, and chlorides inside grouted PT ducts as being the major causes of accelerated corrosion of strands. This corrosion results in tension capacity reduction and can eventually lead to tensile failure of PT strands (NCHRP 1998, ASBI 2000, FDOT 2001a and b, and Hansen 2007). The reduction in tension capacity or tensile failure of PT strands can in turn significantly and adversely affect the safety and serviceability of PT bridges. As these bridges have high importance measures and the consequences of failures are significant, it is important to maintain high levels of safety and serviceability for these PT bridges. According to NCHRP (1998), “there is a pressing need for US bridge engineers to gain an understanding of durability issues associated with segmental construction and to be able to judge on a technical and rational basis the veracity of the on-going moratorium in the UK pertaining to segmental construction....”

Based on the comprehensive research project (TxDOT Project 0-1405), West et al. (1999) provided a state-of-the-art report on durability issues of PT bridge substructures. Schokker et al. (1999) studied the performance of various grout mixtures. Schokker et al. (2002) studied and the corrosion performance of PT strands under various simulated grout environments. Additional research is needed to meet this pressing need and to ensure high levels of safety and serviceability, bridge management authorities are in dire need of tools to quantify the long-term performance of these bridges.

The presence of voids, damages, and openings on PT ducts and anchorages in combination with exposure to severe environmental conditions can result in corrosion of PT strands. This time-variant process can in turn cause reduction in the strength and service reliability indices over time. Therefore, strength and service reliability indices can be considered as quantified measures or indicators for the safety and serviceability, respectively, of PT bridges. The time-variant strength reliability index can be modeled and estimated by using structural reliability techniques and moment capacity and demand models for PT bridges. The time-variant strength reliability index can be modeled and estimated by using structural reliability techniques and in-service stress capacity and demand models for PT bridges. The estimated reliability

indices at future times can then be compared with corresponding target reliability indices. This comparison will help in making decisions on the degree of inspection, repair, and maintenance required. In summary, time-variant structural reliability models can be useful tools to quantify the long-term performance of PT bridges. Probabilistic models for tension capacity of PT strands are necessary to develop time-variant structural reliability models. Experimental data on electrochemical and tension capacity behavior of PT strands are necessary to develop these probabilistic models for tension capacity of PT strands. Unfortunately, the current literature does not provide sufficient information on the electrochemical and tension capacity behavior of PT strands. Furthermore, knowledge on the electrochemical characteristics of ASTM A416 steel when immersed in various cementitious pore solutions and the potential for galvanic corrosion in these systems could assist engineers in developing non-destructive tools for the inspection of corrosion in PT bridges. However, such information is not sufficiently addressed in the literature.

A coupled experimental and analytical program was developed and conducted to fill these knowledge gaps and answer the following questions:

- Can environmental characterization maps be generated to assess corrosion risks in Texas?
- Can information be generated to non-destructively inspect or detect electrochemical corrosion of strands in PT systems?
- What are the critical void, material, environmental, and other factors that can significantly influence the corrosion activity of embedded strands?
- Does this corrosion significantly influence the tension capacity of strands over time? If so, how can the probabilistic tension capacity of strands be modeled and assessed?
- Does the corrosion-induced loss in tension capacity of strands affect the strength and serviceability of PT bridges? If so, how can the strength and service reliabilities of PT bridges be modeled and assessed?

This research will attempt to answer these questions to assist bridge owners in ensuring safe and reliable PT bridges for long durations. The experimental part of this research includes electrochemical and tension capacity testing of PT strands exposed to various exposure conditions. The analytical part of this research includes modeling and assessing of probabilistic

tension capacity of PT strands and modeling and assessing of structural reliability of PT bridges. Both strength and service reliability are modeled as a function of time and other influencing parameters for a typical PT bridge. These reliability models are then used to assess time-variant strength and service reliability of a PT bridge subjected to HS20 and HL93 loading conditions. It is important to note that these models can assess the structural reliability based on climatic conditions and the data on void and damage conditions of bridges while minimizing expensive and non-routine bridge inspections. The objective of this research is to develop general reliability models for PT bridges. Further development of the model will be needed for assessing the reliability of specific PT bridges in Texas. Based on the long-term structural reliability assessments, the inspection, repair, and maintenance programs can be optimized and funds can be appropriately allocated to meet other public needs, while ensuring safe PT bridges.

Volume 2 of this document addresses the procedures, performance of repair materials, inspection and repair procedures, and modeling of economic risk of PT bridges.

4. ENVIRONMENTAL CHARACTERIZATION MAPS OF TEXAS

Equation Section (Next)

4.1. INTRODUCTION

This section provides environmental characterization maps of Texas. These maps can assist engineers in preliminary assessment of corrosion risk of a PT bridge based on its geographic location. In general, an over-specific map could lead engineers to believe that a particular geographic location poses a low-level corrosion risk to PT bridges, while a much higher corrosion risk might actually be present. Therefore, it is important that this map be relied upon only as a guide and not a map providing specific information regarding corrosion activity in tendon systems. These maps do not take into account the corrosion risks due to local exposure conditions (e.g., the presence of moisture, chlorides, voids, and damages on PT systems). The effects of these and other factors are presented in Sections 5, 6, 7, and 8.

The ArcView[®] (version 8.2) software program was used to analyze TxDOT organizational districts in order to assess a corrosion risk level for PT bridge systems. Maps showing environmental factors and their distributions across the US were divided into equal portions and assigned a corrosion risk factor. Arcview[®] 8.2 was then used to trim each of these maps to the TxDOT districts. A questionnaire was sent to each TxDOT district office. Responses obtained from the district engineers were then used to determine the chloride factors for each of the districts. ArcView[®] 8.2 was then used to summarize the chloride and corrosion risk factors for each district. Finally, a qualitative analysis was used to determine the corrosion risk posed to PT bridges in each district in Texas.

4.2. APPROACHES IN DEVELOPING MAPS

The first step in analyzing the environmental factors was to transform the quantitative data from each map into a qualitative scale on a national level. The maps for freeze period, temperature, relative humidity, and rainfall with seven or nine categories were developed (Figure 4-1). However, in each case, the highest and lowest categories often represented extreme portions of

the US and a relatively small area when compared to the other categories. With this in mind, it is easier to divide the remainder of the US into equal portions based on geographic location. In considering all of the maps, it was decided that dividing each category into five divisions would provide a simple method for scaling all of the environmental factors. In each case, the first and last division represents the extreme portions of the US while the three middle divisions represent larger, broader portions of the US (Figure 4-1).

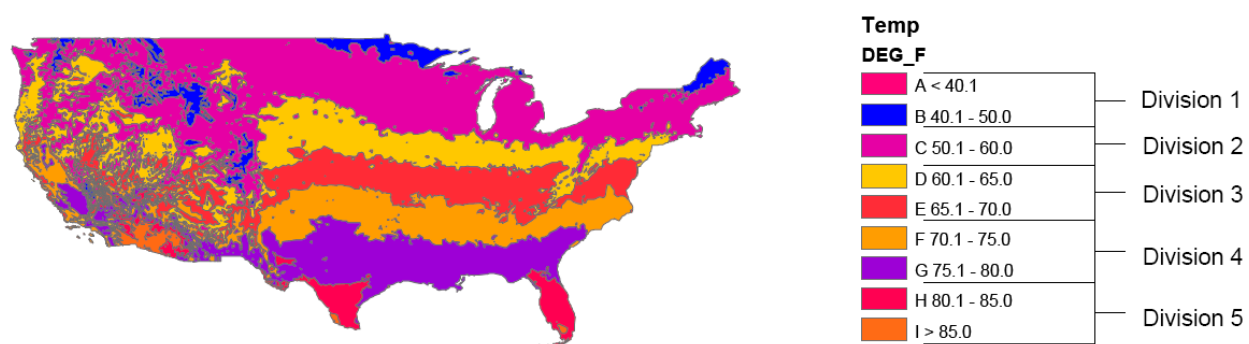


Figure 4-1. Mean Daily Maximum Temperature in the United States.

Here is a brief note on some specifics about the programming in ArcView[®] 8.2.

4.3. FREEZE-DAY, TEMPERATURE, RELATIVE HUMIDITY, AND RAIN-DAY MAPS

Similar procedures were followed for each of the four environmental factors (i.e., freeze period, temperature, relative humidity, and rainfall). After dividing the environmental factors, it was then necessary to assign a numerical value for the corrosion risk to each division so that a function could be created relating all of the factors. For simplicity, it was decided that each division would receive its corresponding numerical value for the corrosion risk (e.g., division one would receive a risk value of one). While this method neglects the fact that the individual factors are not truly weighted the same, it still provides the necessary information for creating a preliminary risk assessment. The task of creating a true, weighted function for each of the factors is beyond the scope of this project. In addition, the purpose of this project is to create a map assessing the corrosion risk factors for the state of Texas on a national scale. Using this approach

will allow the state of Texas to be reviewed on a national scale, which is important to the initial corrosion risk assessment. If the topographic maps were trimmed to the state of Texas first and then the risk factors scaled, the corrosion assessment may over-exaggerate some factors while also neglecting how they compare to the remainder of the nation.

Once each of the environmental factors had been scaled and assigned a corrosion risk level, they were trimmed to the state of Texas using ArcMap™ 8.2 and a shapefile of the TxDOT districts. Once this was completed, the districts were assigned into a category based on where a majority of their area lay. To assist with this division, the districts were intersected with the trimmed environmental maps to determine where the majority of a district's area lay. The maps showing freeze days, temperature, relative humidity, and rainfall are shown in Figure 4-2, Figure 4-3, Figure 4-4, and Figure 4-5, respectively.

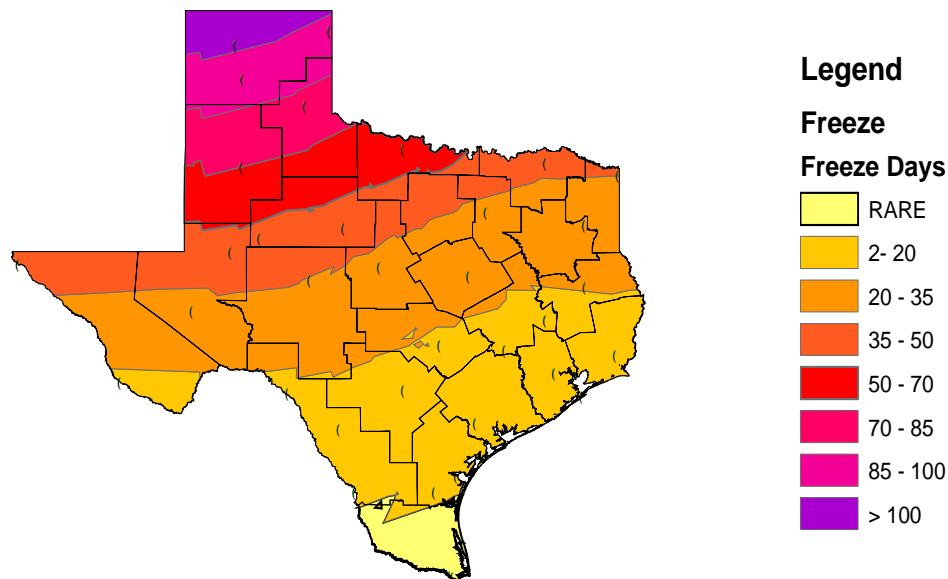


Figure 4-2. Number of Freeze Days in Texas (Data Collected from 1995-2000).

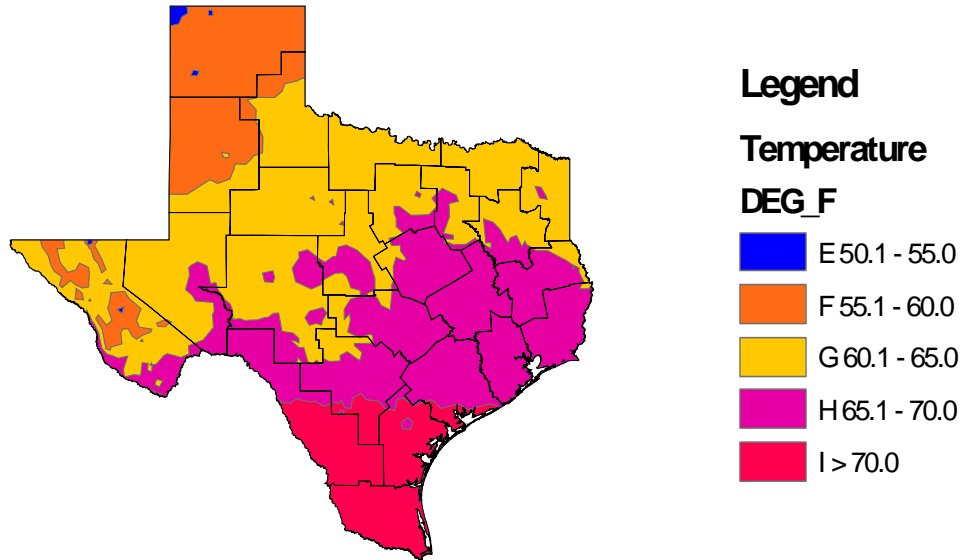


Figure 4-3. Annual Average Temperature in Texas (Data Collected from 1961-1990).

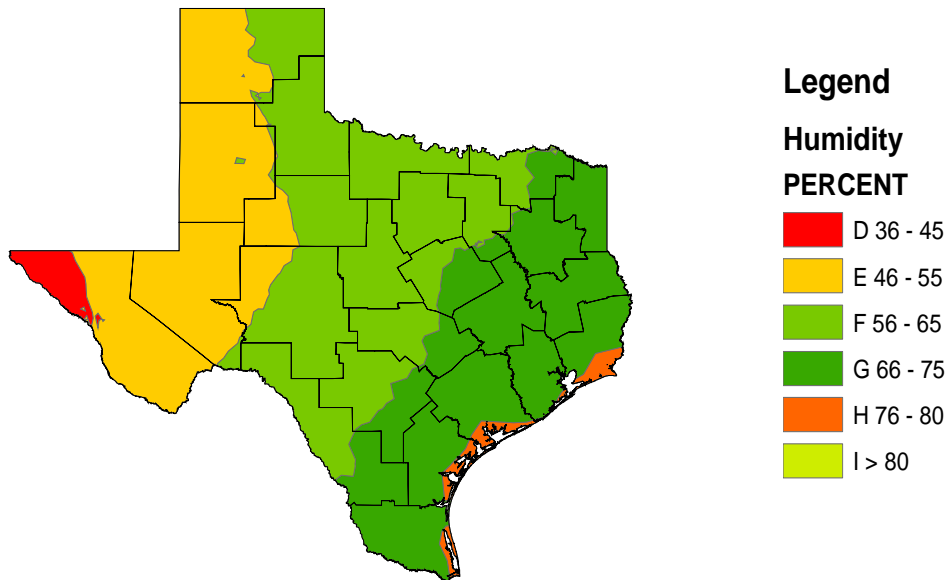


Figure 4-4. Annual Average Relative Humidity in Texas (Data Collected from 1961-1990).

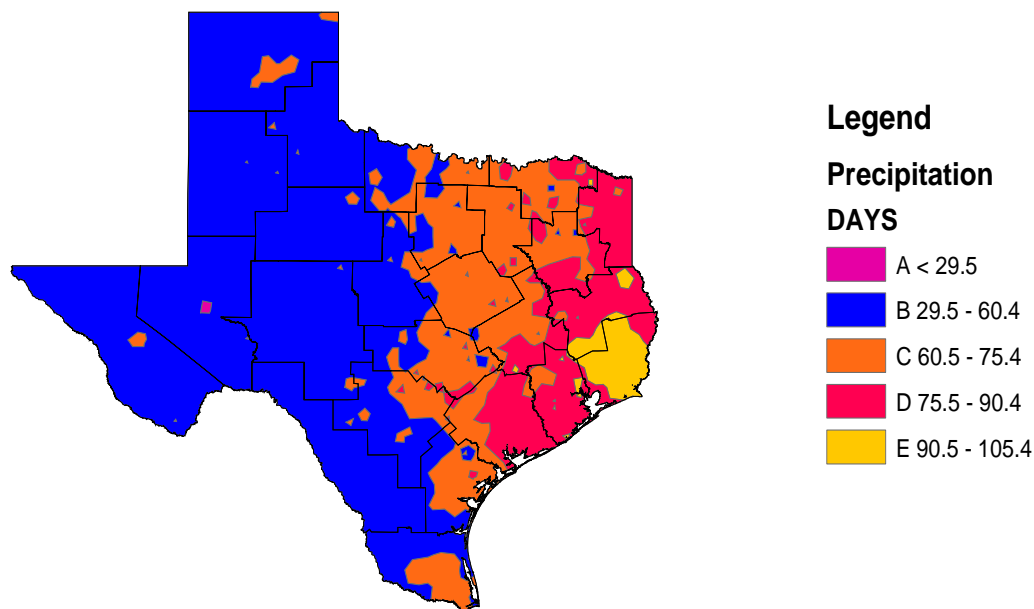


Figure 4-5. Number of Rain-Days in Texas (Data Collected from 1961-1990).

4.4. CHLORIDE MAPS

The information for the chloride factors for the state of Texas was gathered. For each of the 25 TxDOT districts, the district maintenance engineer was contacted via telephone and given the questionnaire. The information gathered from this survey was used to define a chloride usage factor and a chloride frequency factor with both factors ranging from 1–5. The chloride usage factor is a measure of the presence of chlorides for a particular district based on the de-icing measures used by that district or the district’s proximity to the coastline. The chloride frequency factor is a measure of a district’s exposure to chlorides. The two factors must be measured independently and then combined because it is possible for a district to use sodium chloride for de-icing, which is a severe corrosion agent, but to only use this once a year on average. The following distinctions were made concerning the chloride usage and frequency factors, with the divisions being derived from answers to the survey (Tables 4-1 and 4-2).

Table 4-1. Chloride Usage Factor.

Usage Factor	Description
1	Previous use of chlorides
2	Mixed use of aggregate and chlorides
3	Limited use of MgCl or NaCl
4	Consistent use of only chlorides
5	Coastline

Table 4-2. Chloride Frequency Factor.

Frequency Factor	Description
1	Rare usage
2	1–5 times per year
3	6–15 times per year
4	15–30 times per year
5	More than 30 times per year
	Constant exposure (Coastline)

Based on the results from the survey, each district was assigned a chloride usage factor and a chloride frequency factor. These factors were then multiplied to determine a total chloride factor. This total chloride factor was then divided by five in order to scale the chloride factor down to the levels of the environmental factors. For example, if a particular district had limited use of MgCl and applied this approximately 10 times per year, it would be assigned a chloride usage factor of three and a chloride frequency factor of three. Multiplying these factors together and dividing by five gives a total chloride factor of 1.8, which would be rounded up to 2. The total chloride factor distribution for the state of Texas can be seen in Figure 4-6.

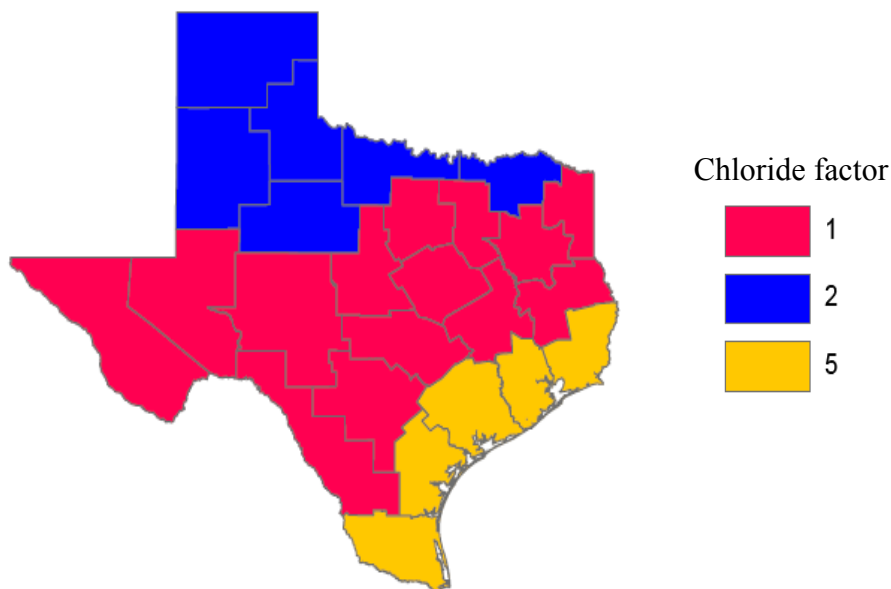


Figure 4-6. Total Chloride Factor.

4.5. TOTAL CORROSION RISK MAPS

All of the individual factors for each of the districts were summed to find a total risk factor for each district. It is important to note that each of the risk factors was weighted equally rather than assigning a weight to each one. The completed maps showing the total corrosion risk factor for each of the TxDOT districts can be seen in Figure 4-7. After finding the total chloride factor for each district, a scale was then developed to compare Texas's districts to the rest of the nation. The scale illustrated in Table 4-3 was used, as it provided a solid, qualitative analysis for the districts. It is interesting to note that the highest corrosion risk factor can be arrived at with 20 points. This is due to the fact that it is unlikely for any place in the US to have extreme corrosion risk factors in every category; furthermore, it is not necessary for a district to have high corrosion risks in every category for a substantial threat to be posed to a PT bridge.

After developing this quantitative risk scale, the districts in Texas were then reassigned to a particular division, and a final qualitative map was created displaying all of the Texas districts and their respective corrosion risk assessments (Figure 4-8). As expected, the coastal regions pose the greatest threat to PT bridges due to the constant exposure to chlorides. It is also

interesting to note that the remainder of the Texas districts fell within a moderate corrosion risk category. This indicates that even with a large state such as Texas, factors concerning post-tensioned bridges are fairly homogenous. This indicates that an analysis could be applied across the United States considering the states as individual entities rather than the smaller transportation districts. The only problem in such an analysis would be the mountain regions where weather patterns can vary significantly across a state.

Table 4-3. Total Corrosion Risk Factors.

Corrosion Risk Factor	Corrosion Risk Assessment
Less than 10	Mid risk
15–11	Moderate risk
16–20	High risk
More than 20	Extreme risk

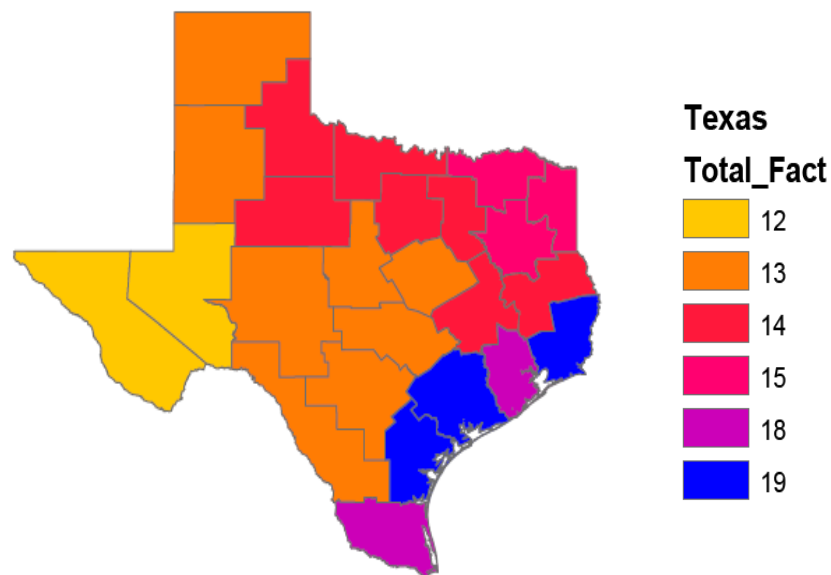


Figure 4-7. Quantitative Assessment of Total Corrosion Risk.

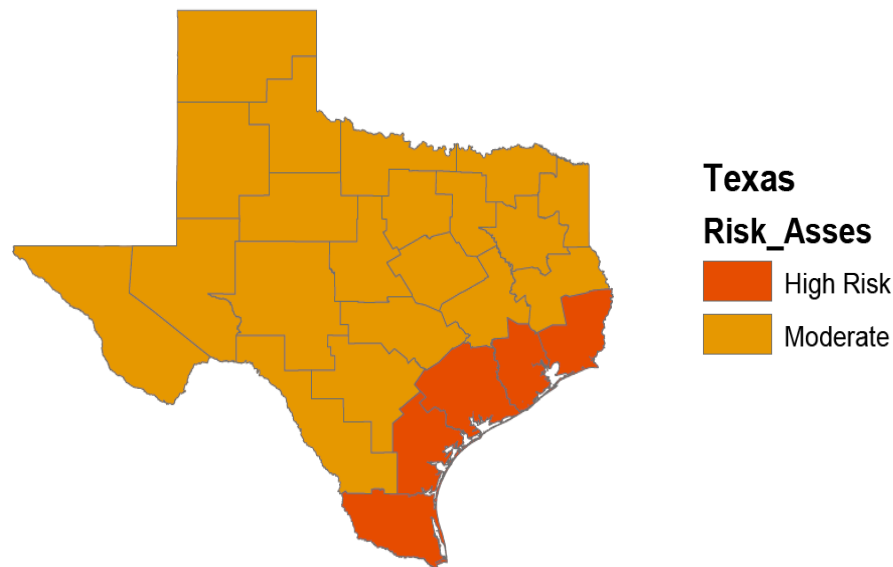


Figure 4-8. Qualitative Corrosion Risk Level.

4.6. SUMMARY

The locations of PT bridges in Texas are provided in Figure 2-17. The majority of PT bridges in Texas lie in a moderate-to-high corrosion risk region. Coastal regions present the highest threat due to consistent exposure to chlorides. The developed environmental characterization map provides valuable insight into the corrosion risk levels across the state of Texas. Additional research could provide valuable insight into how the individual factors could be weighted, providing for a more detailed map. In addition, a better model for combining the chloride exposure and usage factors also needs to be developed, but the procedure for doing this is beyond the scope of this project.

5. EXPERIMENTAL PROGRAM – ELECTROCHEMICAL AND TENSION CAPACITY BEHAVIOR OF WIRES AND STRANDS

Equation Section (Next)

5.1. INTRODUCTION

The major objectives of this research were outlined in Subsection 1.6. To attain these research objectives, a comprehensive experimental and analytical program was developed. Figure 5-1 shows various elements of the experimental program. This section presents the details of the experimental program.

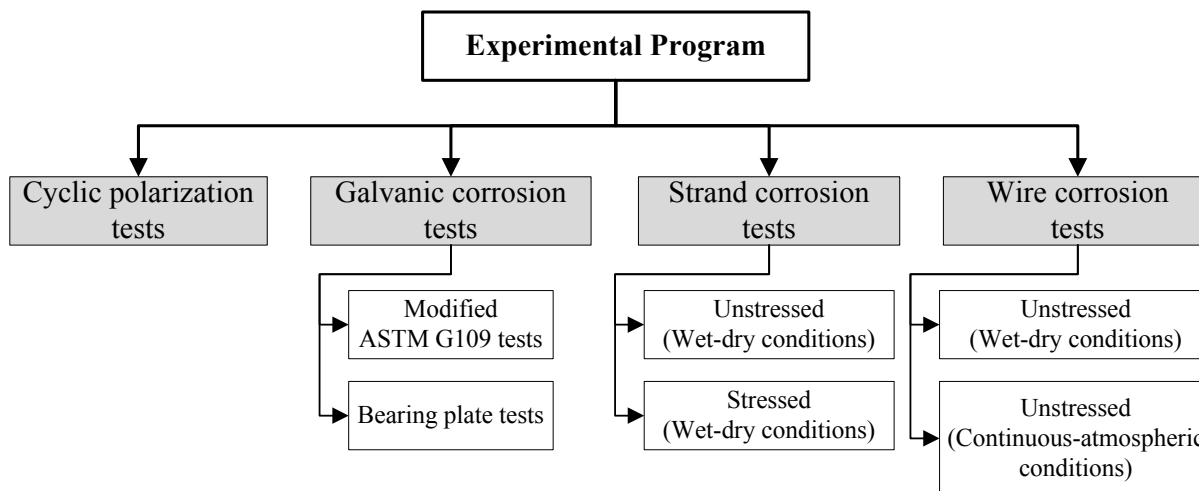


Figure 5-1. Experimental Program (Electrochemical and Tension Capacity Behavior of Wires and Strands).

Results from the cyclic polarization and galvanic corrosion tests are provided in Subsections 6.2 and 6.3, respectively. The tension capacity results from the strand and wire corrosion tests are presented in 6.4 and 6.5. Prior to discussing the objectives, design, layout, and procedures of the experimental program, a discussion on the characteristics of various materials used in the experimental program is provided.

5.2. MATERIALS USED IN THE EXPERIMENTAL PROGRAM

This subsection presents the characteristics of metallic reinforcement, mineral aggregates, cementitious materials, water, exposure solutions, and other materials used to prepare test specimens and perform the corrosion exposure and tension tests.

5.2.1. Metallic reinforcement

Three steel reinforcing material types were used in this research program. Table 5-1 provides the ASTM designations and titles that these reinforcing materials meet and the corresponding notations used in this document. The specific details of these specimens and the test name in which they have been used are provided in the notes below Table 5-1. Further details on how these materials are used are provided in Subsections 5.3 through 5.5. The strands and wires received from the producer/supplier had negligible amount of corrosion and are referred to as “as-received strands” and “as-received wires,” respectively, herein. All the strands and wires used in this test program are from one roll or strand coil.

Table 5-1. Different Steel Reinforcement Types Used in this Research Program.

ASTM designation and title	Notation used in this document
ASTM A416 / A416M-99/A416M-99, Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete	ASTM A416 ¹
ASTM A536-84 (2004), Standard Specification for Ductile Iron Castings	ASTM A536 ²
ASTM A615 / A615M-99, Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement	ASTM A615 ³

1. The 0.6-inch (15 mm) diameter, seven-wire, low-relaxation prestressing strand and the 0.2-inch (5.08 mm) king-wire from this strand used in all the tests.

2. The cast-iron bearing plates used in galvanic corrosion tests.

3. The 0.625-inch (16 mm) diameter, deformed, conventional reinforcement (straight and/or spiral) used in galvanic corrosion tests.

Table 5-2 provides the chemical compositions of these metallic reinforcing materials. The chemical compositions were obtained using Wavelength Dispersive X-ray Spectroscopy following ASTM E 1621-94, *Standard Guide for X-Ray Emission Spectrometric Analysis*, at

15 kV using a calibrated JEOL Superprobe 733. The carbon contents were determined by combustion methods per ASTM E 350-90e1, *Standard Test Methods for Chemical Analysis of Carbon Steel, Low-Alloy Steel, Silicon Electrical Steel, Ingot Iron, and Wrought Iron*, using a LECO C-200 analyzer. The notes below Table 5-2 indicate the test program in which these materials are used.

Table 5-2. Representative Chemical Compositions of Steel Reinforcement Types Used.

Element	ASTM A416 ¹			ASTM A536 ²	ASTM A615 ^{3a}	ASTM A615 ^{3b}
	Sample 1	Sample 2	Sample 3			
C	0.812	0.841	0.842	3.62	0.40	0.38
Mn	0.70	0.811	0.819	0.276	1.03	1.13
P	0.010	< 0.01	0.0001	0.016	0.015	0.012
S	0.010	0.010	0.010	0.009	0.045	0.031
Si	0.25	0.241	0.230	2.61	0.20	0.21
Cu	0.12	0.120	0.115	0.553	0.54	0.46
Ni	0.06	0.050	0.071	0.026	0.22	0.16
Nb	-	0.039	0.022	-	-	-
Mg	-	0.018	0.007	0.046	-	-
Cr	0.03	0.029	0.032	-	0.12	0.14
Mo	0.015	0.0007	0.0008	-	0.073	0.040
Ti	-	0.001	0.003	-	-	-
V	0.069	0.081	0.082	-	0.000	0.002
Sn	0.005	< 0.01	< 0.01	-	0.000	0.014
Al	0.002	0.0003	< 0.01	-	-	-
Cb	0.000	-	-	-	0.000	0.004
N ₂	0.007	0.0436	0.0079	-	-	-
Fe	Remaining	Remaining	Remaining	Remaining	Remaining	Remaining

1. The strands and/or wires used in all the tests; Sample 1 indicates the data from mill report and Samples 2 and 3 indicate the data from laboratory testing.

2. The bearing plates used in the galvanic corrosion tests.

3a. The straight reinforcement used in the galvanic corrosion tests.

3b. The spiral reinforcement used in the galvanic corrosion tests.

Because the nominal cross-sectional area and ultimate tensile strength of “as-received” strands were 0.217 inch² (140 mm²) and 270 ksi (1862 MPa), respectively, the *MUTS* of the “as-received” ASTM A416 strand is 58.6 kips (261 kN). Because of liability concerns, architects, engineers, and designers use *MUTS*_{ARS} for design purposes. The first column (solid diamond

markers) in Figure 5-2 shows the dot plot with observed tension capacities of “as-received” strands (denoted as $C_{T,ARS}$). The second column (solid-square markers) in Figure 5-2 shows the dot plot with tension capacity of “as-received” wires (denoted as $C_{T,ARW}$). In Figure 5-2, the solid-horizontal lines in each column (passing through the data points) indicate the mean values of $C_{T,ARS}$ and $C_{T,ARW}$. The dashed horizontal line at 58.6 kips (261 kN) indicates $MUTS_{ARS}$. As expected, the actual tension capacities are generally found to be higher than $MUTS_{ARS}$. Among the 24 data points, one was found to be 0.1 kip (0.445 kN) less than $MUTS_{ARS}$. Considering the 24 data points, the mean and standard deviation of the $C_{T,ARS}$ was found to be 59.27 kips (263.7 kN) and 0.29 kips (1.3 kN), respectively. The corresponding COV is 0.0049, and it indicates a very small variation.

Six “as-received” wire samples (i.e., the king-wire collected from the “as-received” strands) were tested for tension capacity, and the obtained $C_{T,ARW}$ are shown in the second column in Figure 5-2. Considering all 10 data points, the mean and standard deviation of the $C_{T,ARW}$ was determined to be 9.15 kips (40.7 kN) and 0.05 kips (0.22 kN), respectively. This indicates a very small COV of 0.005 for “as-received” wires and is almost equal to the corresponding COV observed for “as-received” strands. Also, note that ASTM A416 steel exhibited a Rockwell C hardness of 45.

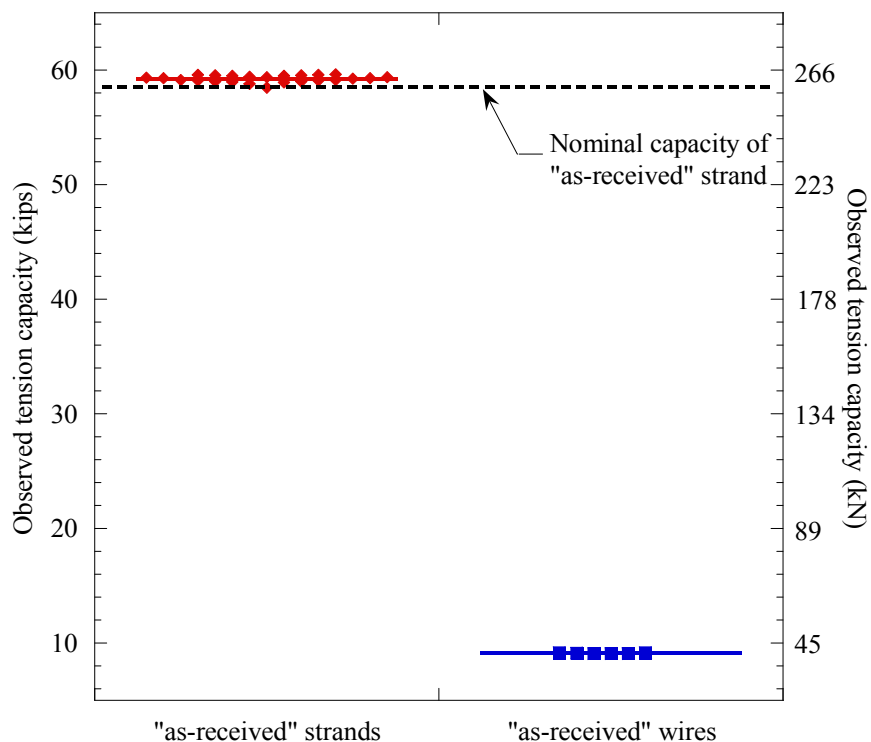


Figure 5-2. Observed Tension Capacities of “As-Received” Strands and Wires.

5.2.2. Mineral aggregates used in concrete

Mineral aggregates were used in the concrete used to make the galvanic corrosion test specimens. Various characteristics of both fine and coarse aggregates were determined using various ASTM standard test procedures, as mentioned later. The particle size distribution curves for the fine and coarse aggregates are shown in Figure 5-3. In Figure 5-3, the three thinner lines (left most) represent fine aggregate, and the three thicker lines (right most) represent coarse aggregate. Also, the solid lines indicate measured values, and the dashed lines indicate the upper and lower limits, as prescribed by ASTM C136, *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates*. Figure 5-3 shows that the particle size distributions of fine and coarse aggregates were within the prescribed upper and lower limits. The fineness modulus of fine aggregate was calculated to be 3.12.

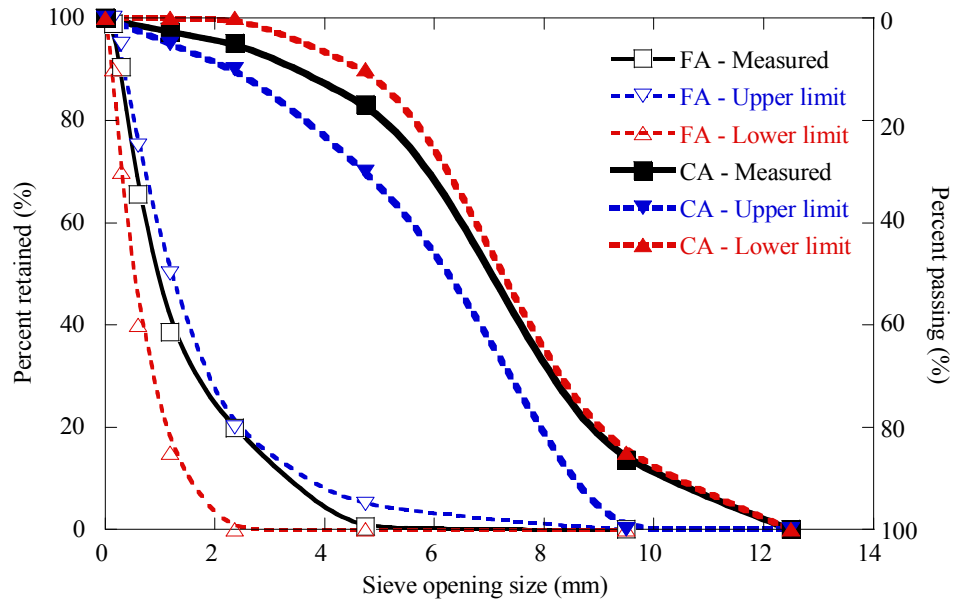


Figure 5-3. Particle Size Distribution Curves for Fine and Coarse Aggregates.

The saturated surface dry (SSD) bulk specific gravity, absorption capacity, and bulk density of the fine and coarse aggregates were determined using ASTM standard test procedures. In addition, the total evaporable moisture content, fineness modulus, and organic impurities content of fine aggregates was also determined using ASTM standard test procedures. Table 5-3 summarizes these test results and the standard test procedures used. These data were used to proportion the concrete mixtures.

Table 5-3. Material Characteristics of Fine and Coarse Aggregates.

Aggregate type	Characteristic property	Quantity	Standard test procedure used
Fine	Bulk specific gravity	2.61	ASTM C128, <i>Standard Test Method for Specific Gravity and Absorption of Fine Aggregate</i>
	Absorption capacity	1.50%	
	Fineness modulus	3.12	
	Moisture content	1.8%	ASTM C566-97, <i>Standard Test Method for Total Evaporable Moisture Content of Aggregate by Drying</i>
Coarse	Bulk specific gravity	2.63	ASTM C127, <i>Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate</i>
	Absorption capacity	1.07%	
	Bulk density	2718 lb/yd ³ (1613 kg/m ³)	ASTM C29, <i>Standard Test Method for Bulk Density ("Unit Weight") and Voids in Aggregate</i>
	Moisture content	2.0%	ASTM C566-97, <i>Standard Test Method for Total Evaporable Moisture Content of Aggregate by Drying</i>

5.2.3. Cementitious materials

Subsection 2.3.2 reviewed different classes of grouts. It was found that Class A, B, and C grouts have been used in most PT bridges in the US. In this test program, Class A grout with a 0.44 water-cement ratio (w/cm) and a commercially available Class C grout (Sikagrout® 300PT) with a 0.27 w/cm were used to prepare the galvanic, strand, and wire corrosion test specimens. The objective of this research was to assess the potential galvanic coupling between different metallic systems in adjacent and different cementitious materials, not that formed by embedding of similar metals in adjacent and different cementitious materials. In addition, Type III cement concrete was also used in preparing the galvanic corrosion test specimens. Table 5-4 shows the chemical compositions of the Type I and III cements used in this test program. Because the Class C grout is proprietary, no chemical analysis was performed on this material. Table 5-5 shows the mixture proportions of the Type III cement concrete and the Class A and C grouts used in this test program.

Table 5-4. Representative Chemical Compositions of Cementitious Materials Used.

Element	Type I cement (used to prepare Class A grout)	Type III cement (used to prepare concrete)
SO ₃	3.16	4.30
SiO ₂	20.74	20.20
Fe ₂ O ₃	1.76	1.60
MgO	1.18	1.70
Al ₂ O ₃	5.12	5.10
as Na ₂ O	0.49	0.12
CaO	64.97	64.00
C ₃ S	61.00	59.00
C ₂ S	-	14.00
C ₃ A	11.00	11.00
C ₄ AF	-	5.00
LOI	1.42	1.80
IR	0.24	0.20

LOI: loss on ignition; IR: insoluble residue.

Table 5-5. Representative Mixture Proportions of Concrete and Grout Used.

Ingredient	Type III cement concrete	Class A grout	Class C grout
	w/cm = 0.40	w/cm or w/p = 0.44	w/p = 0.27
	Batch volume		
	3 ft ³ (0.085 m ³)	2.1 ft ³ (0.059 m ³)	2.1 ft ³ (0.059 m ³)
Water, lb (kg)	38.8 (17.6)	76.1 (34.6)	60.2 (27.4)
Cement or grout powder, lb (kg)	96.9 (44.0)	173.0 (78.6)	223.1 (101.4)
Coarse aggregate*, lb (kg)	151.0 (68.6)	-	-
Fine aggregate, lb (kg)	142.7 (64.9)	-	-

“w/cm” indicates water-cementitious ratio (used only for Type III cement concrete).

“w/p” indicates water-powder ratio (used for Class A and C cementitious grout powder).

* Maximum size aggregate (MSA) = 0.375 inch (9.5 mm).

A total of 10 batches of Type III concrete with a w/cm of 0.40 were prepared to cast the galvanic corrosion test specimens. These concrete batches showed an average slump, measured per ASTM C143, *Standard Test Method for Slump of Hydraulic Cement Concrete*, of 8 inches (20 mm). The average air content, measured per ASTM C231, *Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method*, was 1.7 percent. Following ASTM

C192, *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*, three 4 × 8 inch (10 × 20 mm) concrete cylinders were prepared from each concrete batch. The average 28-day compressive strength of these cylinders was 7.9 ksi (54.4 MPa).

A total of 26 grout batches (13 batches each of Class A and Class C grouts) were prepared and used to cast the strand and wire corrosion test specimens. The water-powder ratios (w/p) of the Class A and C grouts were 0.44 and 0.27, respectively. From the flow cone tests (per ASTM C939-02, *Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete [Flow Cone Method]*), the average efflux times within 1 minute of mixing for the Class A and C grouts were 19 and 23 seconds, respectively. These values fall within the PTI (2003) recommended efflux time limits of 11 to 30 seconds and 5 to 30 seconds for Class A and C grouts, respectively. The average volume of bleed water collected, per ASTM C940-98a, *Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory*, from the Class A and C grout samples were 0.23 and 0 ounces (6.7 and 0 ml), respectively. The average 28-day compressive strengths of 2 × 2 inch cubes of the Class A and C grouts were 6.4 and 7.2 ksi (44.1 and 49.6 MPa), respectively.

5.2.4. Water

The ionic impurities in the water used to prepare the Type III cement concrete, Class A and C grouts, and exposure solutions were determined using an ion-chromatometer (Dionex Model DX-80) and are provided in Table 5-6. Note that the chloride ion concentration (% Cl^-) in the water used to prepare the concrete and grout (first, second, and fifth rows in Table 5-6) was as high as 62 ppm (0.006 %wt or 62 mg/l), whereas the % Cl^- in the water used for other solutions (row 3 and 4 in Table 5-6) was negligible. No ionic impurities were found in the de-ionized water used for cyclic polarization tests.

Table 5-6. Concentrations of Impurities in the Water Used in this Research Program.

Row No.	Type of usage	Usage	Concentration in ppm or mg/l						
			F	Cl	NO ₂	Br	NO ₃	HPO ₄	SO ₄
1	Mixing water	To prepare concrete and grout for galvanic corrosion, unstressed strand and wire corrosion specimens (i.e., standard tap water)	0.188	58.542	-	0.035	0.799	0.469	11.203
2		To prepare grout for stressed strand corrosion specimens (i.e., standard tap water)	0.027	61.777	-	0.086	0.465	-	13.867
3	Exposure solution	For cyclic polarization tests (i.e., de-ionized water)	-	-	-	-	-	-	-
4		For galvanic corrosion and wire corrosion specimens (i.e., distilled water)	0.253	0.711	-	-	-	0.036	-
5		For unstressed and stressed strand corrosion specimens (i.e., standard tap water)	0.027	61.777	-	0.086	0.465	-	13.867

5.2.5. Chloride concentration in the exposure solutions and grouts

In this experimental study, exposure solutions and cementitious grouts with different concentrations of chloride ions are used. The percent chloride concentration in the exposure solutions is denoted as %*sCl*⁻. The percent chloride concentration in the cementitious grout at or near the GAS interface is denoted as %*gCl*⁻.

The exposure solutions with different %*sCl*⁻ were prepared to simulate possible WD exposure conditions in PT bridge elements. Table 5-7 provides the chemical compositions of the exposure solutions used in the cyclic polarization, galvanic corrosion, and strand and wire corrosion tests. In Table 5-7, the first three simulated pore solutions were used as exposure/immersion solutions for cyclic polarization tests. These simulated pore solutions were prepared using the de-ionized water shown in Table 5-6. The 0.0001 and 9 %*sCl*⁻ solutions were used to perform the WD exposure for the galvanic corrosion tests. The 0.0001, 0.006, 0.018, 0.18, and 1.8 %*sCl*⁻ solutions were used to perform WD exposure in strand and wire corrosion test programs.

Table 5-7. Chemical Compositions of Exposure Solutions Used in this Research.

Name of exposure solution	Chemical composition (mg/l)				
	H ₂ O	NaCl	Ca(OH) ₂	NaOH	KOH
Simulated pore solution (0 %sCl ⁻)	966.08	0.00	300	10400	23230
Simulated pore solution (0.06 %sCl ⁻)	965.08	1000.00	300	10400	23230
Simulated pore solution (1.8 %sCl ⁻)	936.08	30000.00	300	10400	23230
0 %sCl ⁻	1000.00	0.00	0	0	0
0.006 %sCl ⁻	990.00	0.10	0	0	0
0.018 %sCl ⁻	999.70	0.30	0	0	0
0.18 %sCl ⁻	997.00	3.00	0	0	0
1.8 %sCl ⁻	970.00	30.00	0	0	0
9 %sCl ⁻	850.00	150.00	0	0	0

The grouts with different %gCl⁻ are attained by immersing the test specimens in exposure solutions with different %sCl⁻ and used in the wire corrosion testing under CA exposure conditions. Further information on %gCl⁻ is provided in Subsection 5.5.4.2.

5.3. CYCLIC POLARIZATION TESTS

5.3.1. Introduction and objectives

Cyclic polarization testing is used to determine electrochemical characteristics, especially the pitting susceptibility, of ASTM A416 steel when immersed in an electrolyte. Results from this test program are provided in Subsection 6.2.

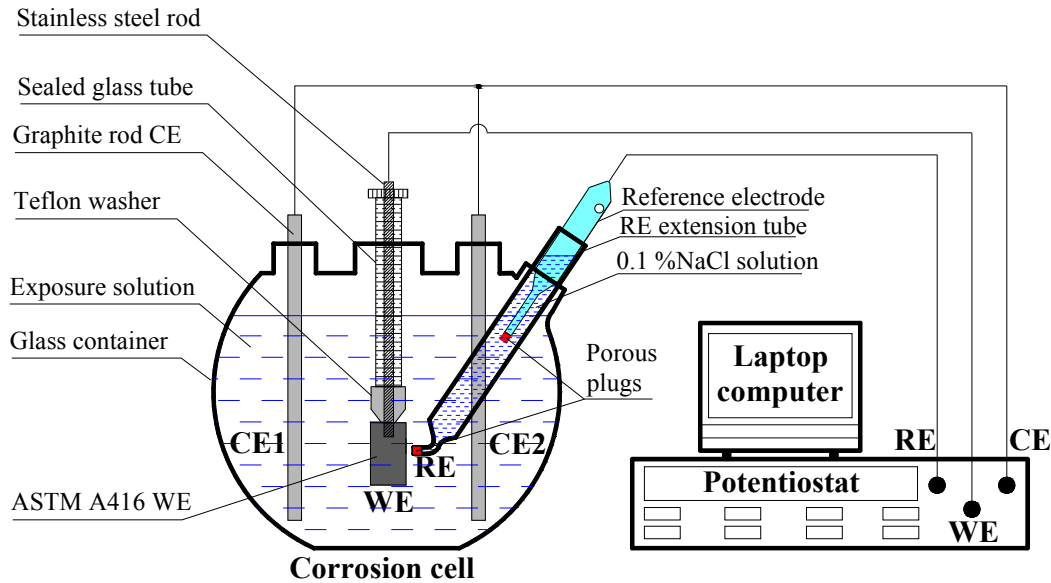
In cyclic polarization testing, a steel specimen is immersed in an electrolyte and kept at a predefined initial potential. This predefined initial potential is more negative to its open circuit potential (OCP), which is defined as the potential at which there is no current (i.e., rest potential or equilibrium potential). This is followed by a scan in the positive direction (i.e., forward scan) through the OCP to a more positive potential. As the potential reaches a predefined maximum value, the direction of the scan is reversed (i.e., reverse scan) and swept through the OCP until the potential reaches the initial potential. The resulting plot shows the relationship between the logarithm of the measured current density and the applied potential. This plot is known as the

cyclic polarization curve. Key electrochemical characteristics that are determined from the cyclic polarization curves and the qualities of the cyclic polarization curves are defined as follows:

- Breakdown potential, E_b : The potential after which the current increases with increasing potential.
- Primary passivation potential, E_{pp} : The potential after which the current either decreases or becomes essentially constant over a finite range of potential.
- Repassivation potential, E_{rp} : The potential corresponding to the lowest current density on the reverse scan of the cyclic polarization curve.
- Passive region: The portion of the forward scan between E_{pp} and E_b .
- Active region: The portion of the forward scan where the potentials are less (more negative) than E_{pp} .
- Transpassive region: The portion of the forward scan where the potentials are larger (more positive) than E_b .
- Positive hysteresis: Occurs when the current corresponding to a particular potential is more in the forward scan than in the reverse scan.
- Negative hysteresis: Occurs when the current corresponding to a particular potential is less in the forward scan than in the reverse scan.

5.3.2. Experimental design and specimen layout

The schematic of the test setup is shown in Figure 5-4. The three major components of the test setup are a corrosion cell, a potentiostat, and a laptop computer. The corrosion cell consists of a steel sample being evaluated, known as working electrode (WE), two graphite counter electrodes (CE), one saturated calomel reference electrode (RE), and electrolytic exposure/immersion solution. The cyclic polarization tests in this research were performed using a potentiostat (Model 1287) manufactured by Solartron, Inc., and the data acquisition and analysis was performed using CorrWare[®] Version 2.80 software developed by Scribner Associates, Inc. The experimental design included cyclic polarization tests of ASTM A416 wire specimens immersed in chloride solutions (i.e., 0, 0.06, and 1.8 %sCT solutions) simulating concrete pore solution.



Notes

CE - Counter electrode (Graphite rod)

RE - Reference electrode (Saturated Calomel electrode with 0.241 V vs SHE)

WE - Working electrode (ASTM A416 steel wire)

Figure 5-4. Schematic of the Cyclic Polarization Test Setup (Note: Not Drawn to Scale).

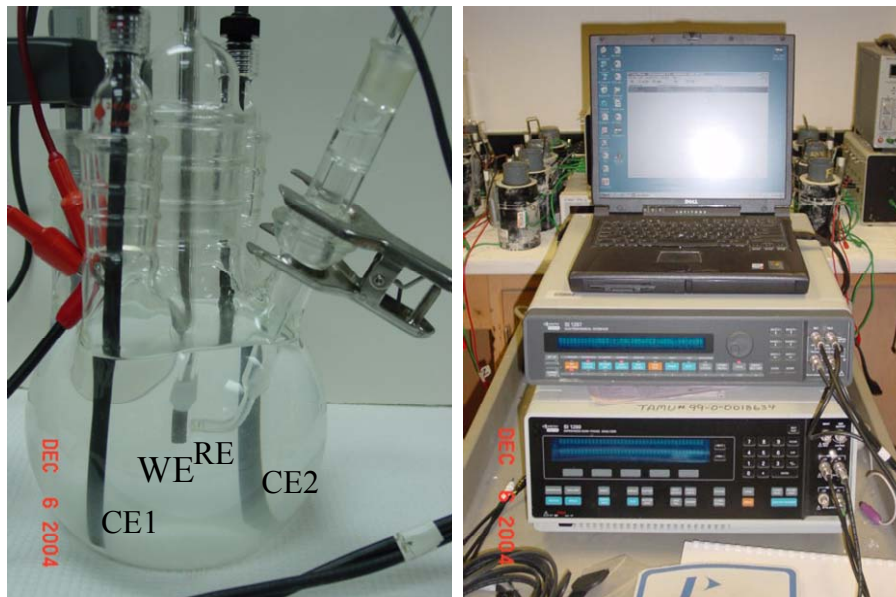


Figure 5-5. Cyclic Polarization Test Setup (Left: Corrosion Cell [Perkin Elmer, Inc.], Right: Potentiostat [Solartron Model 1287], and a Laptop Computer).

5.3.3. Sample preparation and test procedures

The center king-wire from ASTM A416 seven-wire strand was used as the WE. Each cylindrical WE was 0.6 inches (15 mm) in length and 0.2 inches (5.1 mm) in diameter. To minimize the change in surface characteristics due to possible heat generation during the cutting process, the wire specimens were cut using an abrasive cutter with continuous supply of coolant. A 0.08-inch (2 mm) deep hole was then drilled into the specimen and a 3-42 size thread was tapped into one of the end faces. The wire specimens were then immersed in ethyl alcohol and cleaned using an ultrasonic cleaner. The stainless steel threaded rod-Teflon washer system (a part of the corrosion cell kit) was then fastened to the drilled hole on the wire specimen. To ensure uniform current flow at the exposed area and minimize excess current near the edges during the cyclic polarization testing, the end faces and edges of the wire specimens were coated with a thin layer of low-viscosity epoxy (Sikadur[®] 35 Hi-Mod LV). Hence, only the curved surface area (0.325 inch² [210 mm²]) was directly exposed to the simulated pore solution.

A set of three electrochemical tests were performed as follows:

- open circuit potential (OCP) measurement,
- potentiostatic experiment, and
- cyclic polarization (or potentiodynamic scanning).

As soon as the fabricated WE was immersed in the electrolyte, the OCP was measured using the potentiostat for a period of 60 minutes. After WE had attained a stable OCP, the potentiostatic experiment was initiated. The WE was held at an overpotential of -0.100 V versus OCP. The third test in the series was the cyclic polarization. This test was performed at a scan rate of 0.0001 V. The forward scanning potentials ranged from -0.100 V versus OCP to $+1$ V versus OCP. Following the forward scan, a backward scan was initiated, with the final potential being -0.100 V versus OCP. One forward and one backward (or reverse) scan took approximately 10 hours to complete.

As shown in Figure 5-4, the RE was placed inside an inclined RE extension tube filled with 0.1 percent NaCl solution. In Figure 5-4, there are two porous plugs (Vicor[®] frits): one at the tip of the RE and the other at the tip of the RE extension tube. The porous plug at the end of

the RE extension tube was kept at approximately 0.38 to 0.79 inches (1 to 2 mm) from the curved exposed surface of the WE. During the 10-hour cyclic polarization testing, the alkaline exposure solution can partially disintegrate and dissolve the porous plug at the tip of the extension tube. This can cause exchange of solutions on either side of the porous plug. To avoid complete disintegration during the 10-hour long test, a new frit was used for each test. This prevented the leakage of the electrode solution into the exposure solution. Subsection 6.2 presents the results from this test program.

5.4. GALVANIC CORROSION TESTS

5.4.1. Introduction and objectives

In Subsection 2.3.1, a review of the literature on possible galvanic corrosion was provided. To further improve the knowledge on galvanic corrosion in PT systems, the following two galvanic corrosion test programs were developed and conducted: modified ASTM G109 tests and bearing plate tests. The objective of the modified ASTM G109 test program is to test whether galvanic corrosion occurs between the ASTM A615 (conventional) and A416 (prestressing) reinforcement located at the anchorage zones in PT systems and, if so, to assess the increased level of corrosion activity. The objective of the bearing plate test program is to test whether galvanic corrosion occurs between the ASTM A615 and A536 steels located at the anchorage zones in PT systems and, if so, to assess the increased level of corrosion activity. The results from these test programs are provided in Subsection 6.3.

5.4.2. Experimental design and specimen layout – Modified ASTM G109 tests

In PT systems, the ASTM A615 and A416 reinforcing steels are embedded inside Type III cement concrete and grout, respectively. When exposed to moisture and chloride conditions, and if sufficient conductivity exists between these reinforcing steels, galvanic corrosion could occur. To study galvanic corrosion under these conditions, a modified ASTM G109 test specimen was designed. The experimental design (see Table 5-8) is such that the galvanic corrosion between ASTM A416 and A615 steel reinforcements in PT systems that are under negligible ($\approx 0\% \text{ } sCl^-$)

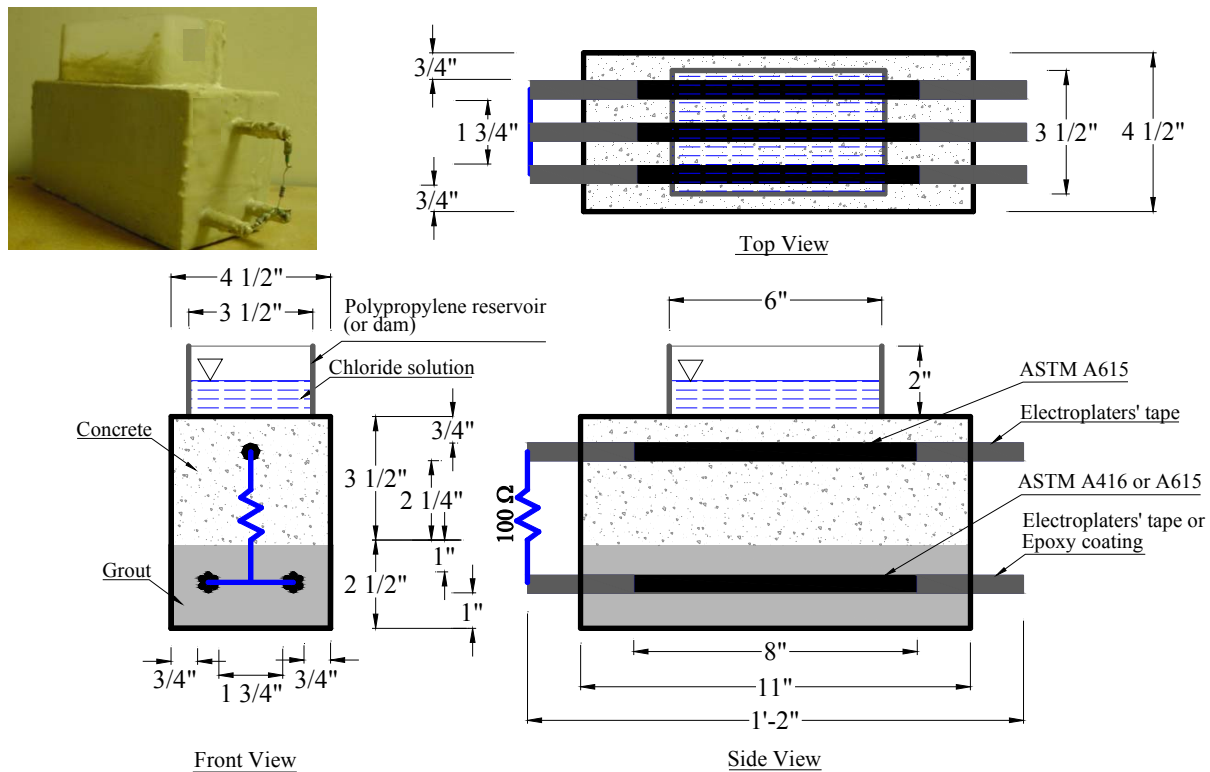
and very high ($\approx 9\% \text{Cl}^-$) chloride exposure conditions can be assessed. A total of 40 test specimens (with 5 specimens for each variable combination) were tested for an exposure period of 10 months.

Table 5-8. Experimental Design for Modified ASTM G109 Test Program.

Concrete type	Grout class	Chloride concentration in the exposure solution (% Cl^-)	Anode	Cathode	Number of samples tested
Type III cement concrete (0.40 w/cm)	A (0.44 w/p)	≈ 0	A615	A615	5*
			A615	A416 (Strand)	5
		≈ 9	A615	A615	5*
			A615	A416 (Strand)	5
	C (0.27 w/p)	≈ 0	A615	A615	5*
			A615	A416 (Strand)	5
		≈ 9	A615	A615	5*
			A615	A416 (Strand)	5

* Indicates control specimens with identical anodic and cathodic material.

Figure 5-6 shows a picture and schematic of the modified ASTM G109 test specimen. Each test specimen consists of an anodic top reinforcement (ASTM A615) embedded in a layer of concrete and two cathodic bottom reinforcement pieces (ASTM A416 or A615) embedded in grout. This provides a cathode-anode ratio of two. When the cementitious material between the anodic (top) and cathodic (bottom) reinforcement is sufficiently conductive, the $100\ \Omega$ electrical resistor connected across the anode and cathode completes the electrical circuit, and this facilitates corrosion measurements. Also, the concrete-grout interface in the test specimens can have an effect on the resulting corrosion activity. To appropriately recognize the effect, if any, of this concrete-grout interface on galvanic corrosion, control specimens with ASTM A615 reinforcements embedded in grout were also tested. Note that no ASTM A615 reinforcement is embedded in grout in PT bridges. In Table 5-8, these control specimens are marked with an asterisk (*). This will facilitate identical effects of concrete-grout interface on corrosion in all the test specimens and enable an appropriate comparative study.



Notes

All dimensions are in inches [1 inch = 2.54 cm].

- ASTM A416 0.6 inch (15 mm) diameter seven-wire strand with epoxy coating at the ends.
- ASTM A615 0.625 inch (16 mm) diameter conventional reinforcement with electroplaters' tape at the ends

Figure 5-6. Photograph and Schematic of a Modified ASTM G109 Test Specimen.

5.4.3. Casting, curing, and exposure procedures – Modified ASTM G109 tests

The modified ASTM G109 test specimens were cast using 0.40 w/cm, Type III cement concrete and Class A and C grouts per the schematic shown in Figure 5-6 and the experimental design in Table 5-8. Both ASTM A615 and A416 steel reinforcements used to cast the specimen were 14 inches (360 mm) long. Prior to embedment, the reinforcement pieces were immersed in an ethyl alcohol bath and cleaned ultrasonically. Electroplaters' tape was then wrapped around the ends (i.e., 3 inches [76 mm] from each end) of the ASTM A615 reinforcement. In the case of ASTM A416 reinforcement, a thin layer of low-viscosity epoxy (Sikadur[®] 35 Hi-Mod LV) was applied around the ends (i.e., 3 inches [76 mm] from each end). These tape-wrappings or

epoxy-coatings were to confine the corrosion to an 8-inch (200 mm) long exposed area near the center of the reinforcement and to avoid corrosion near the ends of the reinforcement pieces. This 8-inch (200 mm) long exposed area lies directly beneath the exposure solution reservoir and has nearly uniform chloride exposure.

When preparing the specimens, the inside surface of the wooden form was coated with a form-release agent. The anodic reinforcement piece was then placed inside the wooden form. The joints/edges were sealed using silicone caulk to prevent leakage of the fluid grout. Following the ASTM C192/C192M, *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*, the concrete was prepared and placed into the wooden forms (up to a height of 3.5 inches [89 mm]) and allowed to cure for one day. The two cathodic reinforcement pieces were then placed inside the wooden forms, and the Class A or C grout was prepared and placed in the remaining height (i.e., 2.5 inches [64 mm]) of the specimen. Following ASTM C192/C192M, the specimens were allowed to cure for an additional 28 days. The wooden forms were removed after 7 days of curing.

After the 28-day curing period, the specimens were allowed to dry at room conditions (approximately 73°F (23°C) and 50 %RH). A rectangular acrylic reservoir (dam) was attached to the center of the top surface, and a 100 Ω electrical resistor was fastened to the reinforcement pieces. A thick layer of high-viscosity epoxy (Sikadur[®] 32 Hi-Mod) was then applied to the concrete surfaces (four sides and the top surface area outside the reservoir) and the visible steel reinforcement surfaces. This was done to prevent moisture exchange through these concrete surfaces, allow one-dimensional diffusion of moisture and chlorides towards the embedded steel reinforcements during the WD exposure, and prevent corrosion at the ends of the reinforcement pieces.

After 14 days of curing, the specimens were moved to an environmental chamber with 100 ±5°F (37.7 ±3°C) temperature conditions. The WD exposure (14 days of ponding followed by 14 days of drying) was initiated on the fifteenth day after the 28-day curing period. To prevent evaporation of exposure solution, the reservoir was covered using a polyethylene sheet. The test specimens were exposed to a total of 11 WD exposure cycles, and corrosion

measurements were made during the exposure. The corrosion measurement and evaluation procedures are provided in Subsection 5.4.6.

5.4.4. Experimental design and specimen layout – Bearing plate tests

In PT systems, the ASTM A536 cast-iron trumpets or bearing plates are surrounded by Type III concrete and filled with Class A or C grout. The concrete surrounding the bearing plates experiences very large bursting forces. To sustain and distribute these large bursting forces, the spiral reinforcement (commonly known as bursting reinforcement) is provided around the bearing plates. These metallic and cementitious materials could be exposed to moisture and chloride conditions. To simulate these conditions and evaluate the magnitude of the galvanic corrosion, the bearing plate test specimen was designed. The experiment was designed to specifically assess the galvanic corrosion between the bearing plate and conventional spiral reinforcement in PT systems that experience WD exposure to $\approx 0\% \text{Cl}^-$ and $\approx 9\% \text{Cl}^-$ solutions. Table 5-9 provides the experimental design for the bearing plate test program. A total of 20 test specimens (with five specimens for each variable combination) were tested for an exposure period of 11 months.

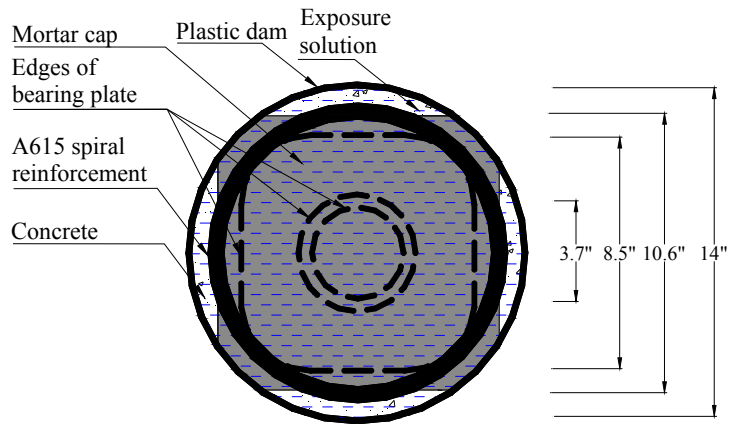
Figure 5-6 shows the photographs of the partially and completely cast bearing plate specimen and its schematic drawings. As shown in Figure 5-7, each test specimen consists of an ASTM A536 bearing plate and ASTM A615 reinforcement. To simulate the field conditions, the ASTM A615 reinforcement was spiraled (at a nominal diameter equal to 10.6 inches [300 mm]) and placed around the ASTM A536 cast-iron bearing plate, the top surface of which is covered with a 0.5 w/cm mortar. The spiraled ASTM A615 reinforcement had 1.7 turns, providing a total surface area of approximately 87.5 inch^2 (565 cm^2). The total surface area of the ASTM A536 bearing plate (model EC-6-7 manufactured by VSL International, Inc.) was 175 inch^2 (1129 cm^2). Therefore, the ratio between the surface areas of the bearing plate and spiral reinforcement was two, which is similar to that between the anodic and cathodic reinforcement pieces in the modified ASTM G109 test specimens. A 100Ω resistor was connected across the bearing plate and the spiral reinforcement to measure electrical current.

Table 5-9. Experimental Design for Bearing Plate Test Program.

Concrete type	Grout class	Chloride concentration in the exposure solution (%sCl ⁻)	Anode	Cathode	Number of samples tested
Type III cement concrete with 0.40 w/cm	A (0.44 w/cm)	≈ 0	A615	A536 (Cast iron)	10
		≈ 9	A615	A536 (Cast iron)	10



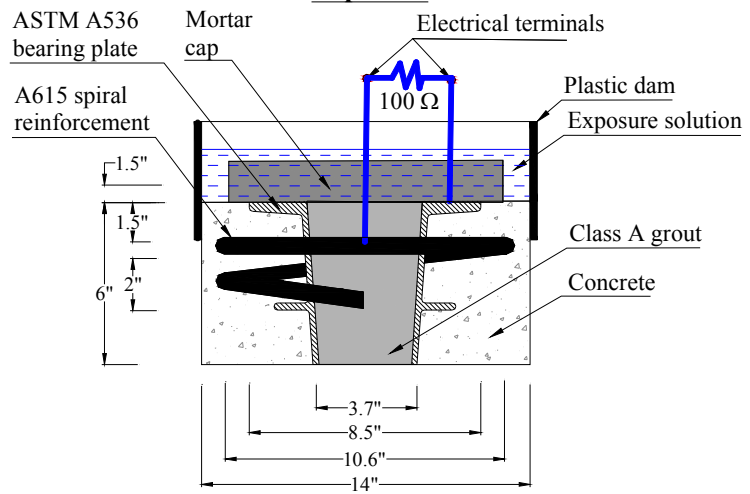
Partially-cast specimen showing the bearing plate and spiral reinforcement



Top View



Completely-cast specimen showing the plastic dam and 100 Ω resistor (inside ellipse), and epoxy coated concrete surface



Front View

Figure 5-7. Photographs and Schematics of a Bearing Plate Test Specimen.

5.4.5. Casting, curing, and exposure procedures – Bearing plate tests

Figure 5-7 shows the schematic of the bearing plate test specimen. These specimens were cast using Type III cement concrete, Class A grout, and ASTM A536 and A615 reinforcement pieces. The experimental design was provided earlier in Table 5-8. The casting procedure is as follows. A 12-inch (300 mm) long electrical lead wire was fastened to one end of the spiral reinforcement. Another similar electrical lead wire was fastened to a threaded hole on the flat surface of the bearing plate. A 14-inch (356 mm) diameter cardboard form was then placed on a flat and clean surface. The spiral reinforcement was then placed inside the circular cardboard form. Following this, the bearing plate was placed (the end with smaller diameter was facing downward) inside the form. Per ASTM C192/C192M, the Type III concrete was then prepared and placed in three layers into the cardboard form and allowed to cure for one day. On the second day, without removing the specimen from the curing room, the conical frustum inside the bearing plate was filled with Class A grout, the top surface of the specimen was covered with a 1.5-inch (38 mm) thick mortar layer, and the cardboard form was removed. The specimens were then cured for an additional 28 days.

After the 28-day curing period, the specimens were moved to a laboratory area with room conditions (approximately 73°F [23°C] and 50 %RH). The curved surface of the specimens was then coated with high-viscosity epoxy (Sikadur[®] 32 Hi-Mod). This was done to prevent moisture exchange through these concrete surfaces and allow one-dimensional diffusion of moisture and chlorides towards the embedded steel reinforcement during the WD exposure. A circular reservoir (dam) was prepared using a flexible and 0.08-inch (2 mm) thick plastic sheet. To prevent leakage of exposure solution, the gap between the concrete and the plastic sheet was sealed using waterproof silicone caulk. A 100 Ω electrical resistor was fastened to the free ends of the two lead wires.

After 14 days of curing, the specimens were moved to an environmental chamber with 100 ±5°F (37.7 ±3°C) temperature conditions, and as in the modified ASTM G109 test, the WD exposure (14 days of ponding followed by 14 days of drying) was initiated. To prevent evaporation of the exposure solution, the reservoir was covered using a polyethylene sheet. The

WD exposure continued for a period of 11 months. Corrosion measurements were made during each WD cycle, per the procedures provided in Subsection 5.4.6.

5.4.6. Corrosion evaluation procedures

The corrosion evaluation procedures for both the modified ASTM G109 and bearing plate tests are identical and based on the ASTM G109 standard practice. On the seventh day of ponding during each WD cycle (say, i^{th} WD cycle), the voltage, V_i , across the 100 Ω resistor is measured. Using Ohm's law, the current, I_i , is determined as follows:

$$\text{Ohm's law: } I_i (\text{Amps}) = \frac{V_i (\text{volts})}{R (\text{ohms})} = \frac{V_i}{100 \Omega} \quad (5.1)$$

The total electrical charge, EC_i , passed from the first WD cycle to the i^{th} WD cycle can be determined using the following equation provided in the ASTM G109:

$$EC_j (\text{Coulombs}) = \underbrace{EC_{j-1}}_{\text{Coulombs}} + \left[\underbrace{(t_j - t_{j-1})}_{\text{Seconds}} \times \underbrace{(I_j - I_{j-1})}_{\text{Amps}} / 2 \right] \quad (5.2)$$

This calculated total corrosion indicates the severity of ongoing corrosion in each test specimen. The results of these galvanic corrosion tests are provided in Subsection 6.3.

5.5. STRAND AND WIRE CORROSION TESTS

5.5.1. Introduction and objectives

Subsection 2.2 presented various case histories on failure of strands in PT bridges. Subsection 2.7.1.1 noted that the tension capacity of strands is a critical parameter that directly influences the structural capacity of PT bridges. When exposed to various corrosive conditions, the tension capacity of strands can decrease as a function of time. However, no deterministic or

probabilistic models exist to predict the time-variant tension capacity of PT strands in tendons that are exposed to various void, material, environmental, stress, and other exposure conditions. The strand and wire corrosion tests were developed and performed to obtain the data necessary to develop probabilistic tension capacity models for PT strands. These tension capacity models are presented in Section 7 and are used in the modeling and assessment of time-variant structural reliability of PT segmental bridges presented in Section 10.

The main objectives of the strand and wire corrosion tests are to generate the data necessary to develop probabilistic models for the tension capacity (C_T) of strands that are exposed to both WD and CA conditions. These models are developed in Sections 7, 8, and 9. A total of 536 and 278 strand and wire specimens, respectively, were tested. Experimental designs and specimen layouts for the strand and wire corrosion tests are provided next.

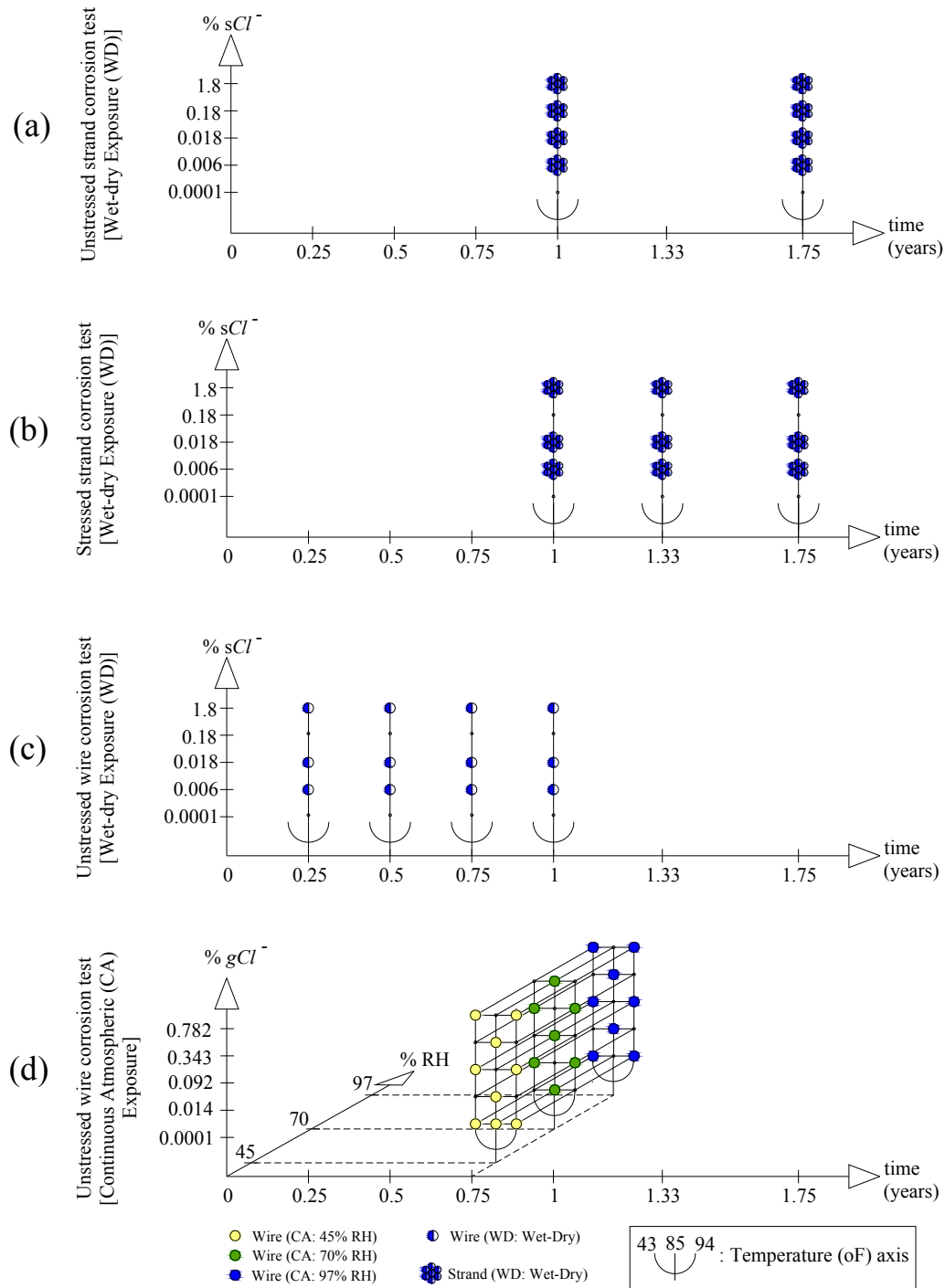
5.5.2. Experimental design and specimen layout

The following four strand/wire corrosion test programs were performed:

1. unstressed strand corrosion test – WD exposure conditions (374 specimens),
2. stressed strand corrosion test – WD exposure conditions (162 specimens),
3. unstressed wire corrosion test – WD exposure conditions (64 specimens), and
4. unstressed wire corrosion test – CA exposure conditions (61 specimens).

Figure 5-8 (a), (b), (c), and (d) show the experimental designs (in graphical format) for these four test programs, respectively. The first three test programs are under WD exposure conditions, and the fourth test program is under CA exposure conditions. The half-filled, seven-circle markers and half-filled, single-circle markers indicate strand and wire specimens, respectively, under WD exposure conditions. The completely filled single-circle markers indicate wire specimens under CA exposure conditions. More detailed information of these four test programs is provided later. The factors and levels of test parameters have been designed such that statistically valid and field relevant tension capacity results of strands and wires can be obtained. Subsections 5.5.2.1 and 5.5.2.2 discuss the experimental designs and layouts of the

strand and wire corrosion tests, respectively. The full data set of results from this experimental program is provided in Appendix A.



Note:
Tension capacities of "as-received" strands and wires with negligible corrosion were also determined.

Figure 5-8. Experimental Designs for All the Strand and Wire Corrosion Tests.

5.5.2.1. Strand corrosion tests

Figure 5-8 (a) and (b), Table 5-10, and Table 5-11 provide the experimental designs for the unstressed and stressed strand corrosion tests. A total of 374 and 162 unstressed and stressed strand corrosion specimens, respectively, were tested. The effects of the qualitative parameters void type (VT), grout class (GC), and f_{axial} and quantitative parameters MC , $\%sCl^-$, and WD exposure time (t_{WD}) on strand corrosion and resulting loss in tension capacity were tested.

Table 5-10. Experimental Design Showing the Number of Test Specimens in the Unstressed Strand Corrosion Test under WD Exposure Conditions*.

Grout class, GC	Wet-dry exposure time, t_{WD} , months	Moisture content, MC	Chloride ion concentration in exposure solution, $\%sCl^-$	Void type (VT)				
				No void (NV)	Parallel void (PV)	Orthogonal void (OV)	Inclined void (IV)	Bleed water void (BV)
A	12	Low	0.0001**	5	5	5	5	5
		High	0.006	5	5	5	5	5
			0.018	5	5	5	5	5
			0.18	5	5	5	5	–
			1.8	5	3	3	5	5
	21	Low	0.0001**	5	5	5	5	5
		High	0.006	5	5	5	5	5
			0.018	5	5	5	5	5
			0.18	5	5	5	5	–
			1.8	5	4	4	5	5
C	12	Low	0.0001**	5	5	5	5	5
		High	0.006	5	5	5	5	5
			0.018	5	5	5	5	5
			0.18	5	5	5	5	–
			1.8	5	5	5	5	5
	21	Low	0.0001**	5	5	5	5	5
		High	0.006	5	5	5	5	5
			0.018	5	5	5	5	5
			0.18	5	5	5	5	–
			1.8	5	5	5	5	5
Total number of unstressed strand specimens for each VT				80	77	77	80	60
Total number of unstressed strand specimens				374				

* 24 “as-received” strand specimens with negligible corrosion were also tested.

** Control specimens that are exposed to standard room conditions without WD cycles.

– Indicates no data available.

Table 5-11. Experimental Design Showing the Number of Test Specimens in the Stressed Strand Corrosion Test under WD Exposure Conditions*.

Grout class, GC	Wet-dry exposure time, t_{WD} , months	Moisture content, MC	Chloride ion concentration in exposure solution, %sCl ⁻	Void type (VT)		
				No void (NV)	Parallel void (PV)	Orthogonal void (OV)
A	12	Low	0.0001**	–	–	2
		High	0.006	3	3	3
			0.018	3	3	3
			1.8	3	3	3
	16	Low	0.0001**	–	–	2
		High	0.006	–	3	3
			0.018	–	3	3
			1.8	–	3	3
	21	Low	0.0001**	–	–	2
		High	0.006	3	4	4
			0.018	3	4	4
			1.8	3	4	4
C	12	High	0.006	3	3	3
			0.018	3	3	3
			1.8	3	3	3
	16	High	0.006	–	3	3
			0.018	–	3	3
			1.8	–	3	3
	21	High	0.006	3	4	4
			0.018	3	4	4
			1.8	3	4	4
Total number of stressed strand test specimens				36	60	66
Grand total number of stressed strand test specimens				162		

* 24 “as-received” strand specimens with negligible corrosion were also tested.

** Control specimens that are exposed to standard room conditions without WD cycles.

– Indicates no data available.

Figure 5-9 shows the schematics of the strand corrosion test specimens. Each strand test specimen was made of a 41-inch (1041 mm) long strand piece meeting the ASTM A416 specifications. Five VTs that are intended to represent typical geometries of GAS interfaces found in PT bridges were identified. These VTs included no void (NV), parallel void (PV), orthogonal void (OV), inclined void (IV), and bleed water void (BV) and are defined as follows:

- NV (see Figure 5-9[a]): No voids are present and the tendons are fully grouted. The 12-inch (300 mm) long strand surface beneath the reservoir (shown as “R” in Figure 5-9[a]) has a 0.25-inch (6 mm) grout cover. (Although this indicates a condition without any voids, NV is considered

as a level of the qualitative variable *VT* for convenience of presentation in this document, herein.)

- PV (see Figure 5-9[b]): The longitudinal axis of the partially embedded strand is parallel to the grout surface. The 12-inch (300 mm) long strand surface beneath the reservoir (shown as “R” in Figure 5-9[a]) is in direct contact with the exposure solution (i.e., no grout cover). This void condition may be found in the mid-span region of PT girders with a horizontal tendon profile.
- OV (see Figure 5-9[c]): The longitudinal axis of the partially embedded strand is orthogonal to the grout surface. This void condition may occur in PT columns or piers or other elements with a vertical profile. In addition, depending on the flow characteristics of the fresh grout, the OV type may also be found in PT ducts with horizontal or inclined profiles.
- IV (see Figure 5-9[d]): The longitudinal axis of the partially embedded strand is at a 45° angle to the grout surface. This represents a void condition that may be found near anchorage zones of PT girders with an inclined tendon profile.
- BV (see Figure 5-9[e]): This void condition can form due to the evaporation of bleed water. In the test specimens, this is represented by a 5-inch (127 mm) long, thin grout layer on the strand surface near the GAS interface (shown as “GS” in Figure 5-9[e]). This thin grout covering is the only difference between the BV and IV samples. Note that in the field, the length of such thin grout layers formed due to the formation and evaporation of bleed water could be much larger than 5 inches (127 mm).

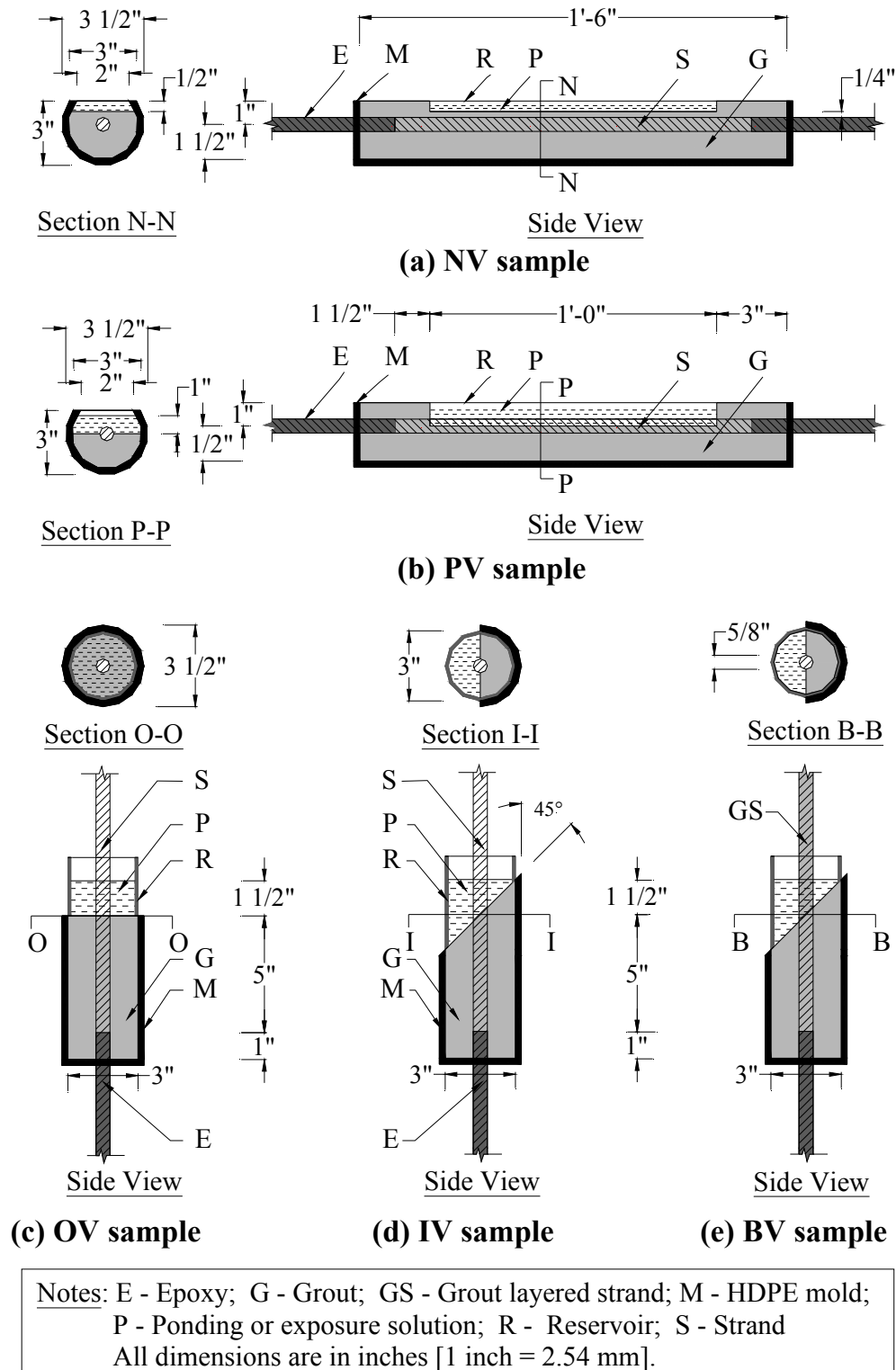


Figure 5-9. Schematics of Strand Corrosion Test Specimens.

To test the effect of the qualitative parameter GC on tension capacity, an approximately equal number of test specimens were cast with Class A and C grouts.

An f_{axial} level of 0, or approximately 150 ksi (0 or 1034 MPa), were applied to the test specimens and were denoted as unstressed and stressed strand specimens, respectively. IV and BV samples were not included in the stressed strand corrosion test program. Three stressed strand specimens were tested in each parameter combination, as compared to five unstressed strand specimens.

Specimens *with* and *without* WD exposure were considered to be under *high* and *low* MC (see third columns of Table 5-10 and Table 5-11). The chloride conditions tested include CA environment with negligible chloride concentration and WD environments with 0.006, 0.018, 0.18, and 1.8 % sCl^- solutions. For each test specimen, the strand surface area exposed to the grout and exposure solution can be considered as the cathode and anode regions, respectively (e.g., in the OV samples, the length of the portions of the strand exposed to grout and exposure solution are 5 and 1.5 inches (127 and 38 mm), respectively, resulting in a cathode-to-anode ratio of 5:1.5). Using this, the cathode-to-anode ratios on the OV, IV, BV, and PV samples were approximately 5:1.5, 5:1.5, 5:1.5, and 2:1, respectively.

As shown in Table 5-10 and Table 5-11, the total duration of the strand corrosion test programs under WD exposure conditions was 21 months. In particular, the unstressed strand specimens were exposed for 0, 12, and 21 months, and the stressed strand specimens were exposed for a period of 0, 12, 16, and 21 months.

5.5.2.2. *Wire corrosion tests*

Figure 5-8(c) and Table 5-12 show the experimental designs for the unstressed wire corrosion test program under WD exposure conditions. Figure 5-8(d) and Table 5-13 show the experimental design of the unstressed wire corrosion test program under CA exposure conditions. A total of 125 wire specimens were exposed to WD (64 specimens) and CA (61 specimens) exposure conditions for different test periods. At the end of each test period, the corroded wire specimens were tested for the residual tension capacity. The quantitative test

parameters included in the WD test program were t_{WD} and $\%sCl^-$. The quantitative parameters included in the CA test program were t_{CA} , RH , T , and $\%gCl^-$. Note that all the wire specimens were prepared using Class A grout and were not tested under stressed conditions. Also, only the effect of OV condition on wire capacity was tested in this program.

Table 5-12. Experimental Design Showing the Number of Test Specimens in the Unstressed Wire Corrosion Test under WD Exposure Conditions*.

Void type, <i>VT</i>	Grout class, <i>GC</i>	Wet-dry exposure time, t_{WD} , months	Chloride ion concentration of the exposure solution $\%sCl^-$				
			0.0001	0.006	0.018	0.18	1.8
OV	A	3	3	3	3	3	3
		6	3	3	3	4	4
		9	3	3	3	4	4
		12	3	3	3	3	3
Total number of WD wire test specimens			12	12	12	14	14
Grand total number of WD wire test specimens			64				

* 6 “as-received” wire specimens with negligible corrosion were also tested.

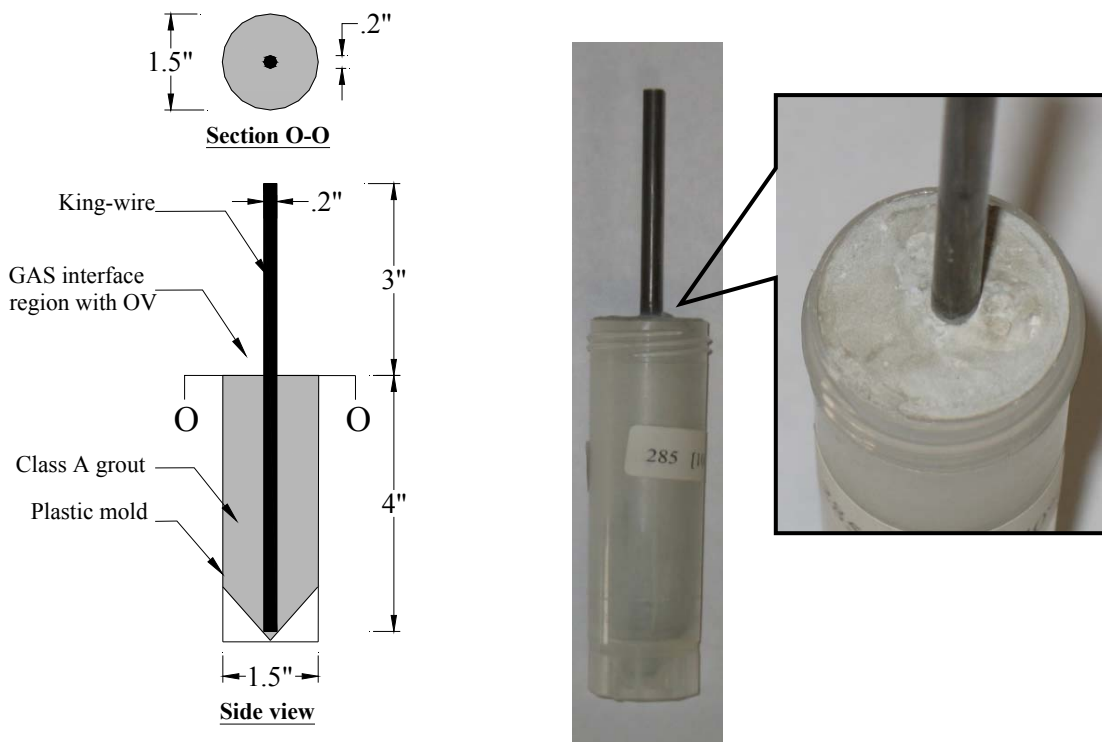
Table 5-13. Experimental Design Showing the Number of Test Specimens in the Unstressed Wire Corrosion Test under CA Exposure Conditions*.

Void type, <i>VT</i>	Grout class, <i>GC</i>	Continuous-atmospheric exposure time, <i>t_{CA}</i> , months	Ambient relative humidity <i>RH</i> (%)	Ambient temperature, <i>T</i> , °F (°C)	Chloride ion concentration in the grout at or near GAS interface, % <i>gCl</i> ⁻ , %wt cement (lb/yd ³)				
					0.00001 (0.0001)	0.014 (0.13)	0.092 (0.87)	0.343 (3.25)	0.782 (7.4)
OV	A	9	45	43 (6.1)	2	–	3	–	4
				85 (29.4)	2	3	–	4	–
				94 (34.4)	2	–	3	–	4
			70	43 (6.1)	–	3	–	4	–
				85 (29.4)	3	–	4	–	4
				94 (34.4)	–	3	–	4	–
			97	43 (6.1)	3	–	4	–	4
				85 (29.4)	–	3	–	4	–
				94 (34.4)	3	–	4	–	4
Total number of CA wire test specimens					15	12	18	16	20
Grand total number of CA wire test specimens					61				

* 11 “as-received” wire specimens with negligible corrosion were also tested.

– Indicates that no data is available.

Figure 5-10 shows the schematic and photograph of a wire corrosion test specimen with an OV condition. The wire corrosion test specimens were prepared using 7-inch (178 mm) long center king-wires removed from ASTM A416 strands.



Note: All dimensions are in inches.
1 inch = 2.54 mm.

Photographs of a wire corrosion test specimen with an orthogonal void

Figure 5-10. Schematic and Photograph of Wire Corrosion Test Specimen.

As shown in Figure 5-8(c) and Table 5-12, the wire specimens under WD exposure conditions were exposed for a period of 3, 6, 9, and 12 months, respectively. As shown in Figure 5-8(d) and Table 5-13, the wire specimens under CA exposure conditions were tested only at 9 months.

For the WD test program, the five levels of % sCl^- were 0.0001, 0.006, 0.018, 0.18, and 1.8 and match with those in the strand corrosion test program. For the CA test program, the five levels of % gCl^- were 0.00001, 0.014, 0.092, 0.343, and 0.782 percent by weight of cement (0.0001, 0.13, 0.87, 3.25, and 7.4 lb/yd³). In the probabilistic models developed in this document, the parameter % gCl^- will refer to the quantities in percent by weight of cement.

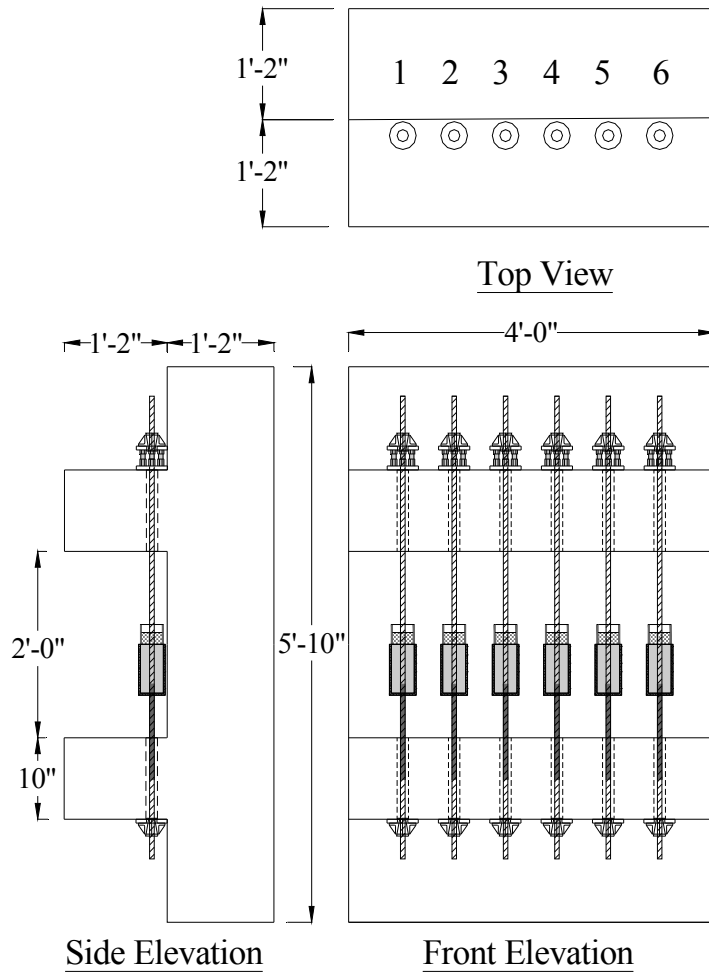
Two additional quantitative parameters included in the experimental design for the CA test program were RH and T . Three levels of RH were considered and are 45, 70, and 97 percent.

Three levels of temperature (T) considered were 43, 85, and 94°F (6.1, 29.4, and 34.4°C) representing cold, moderate, and hot climates.

5.5.3. Concrete reaction frames and stressing operations for stressed strand specimens

5.5.3.1. Concrete reaction frames and layout

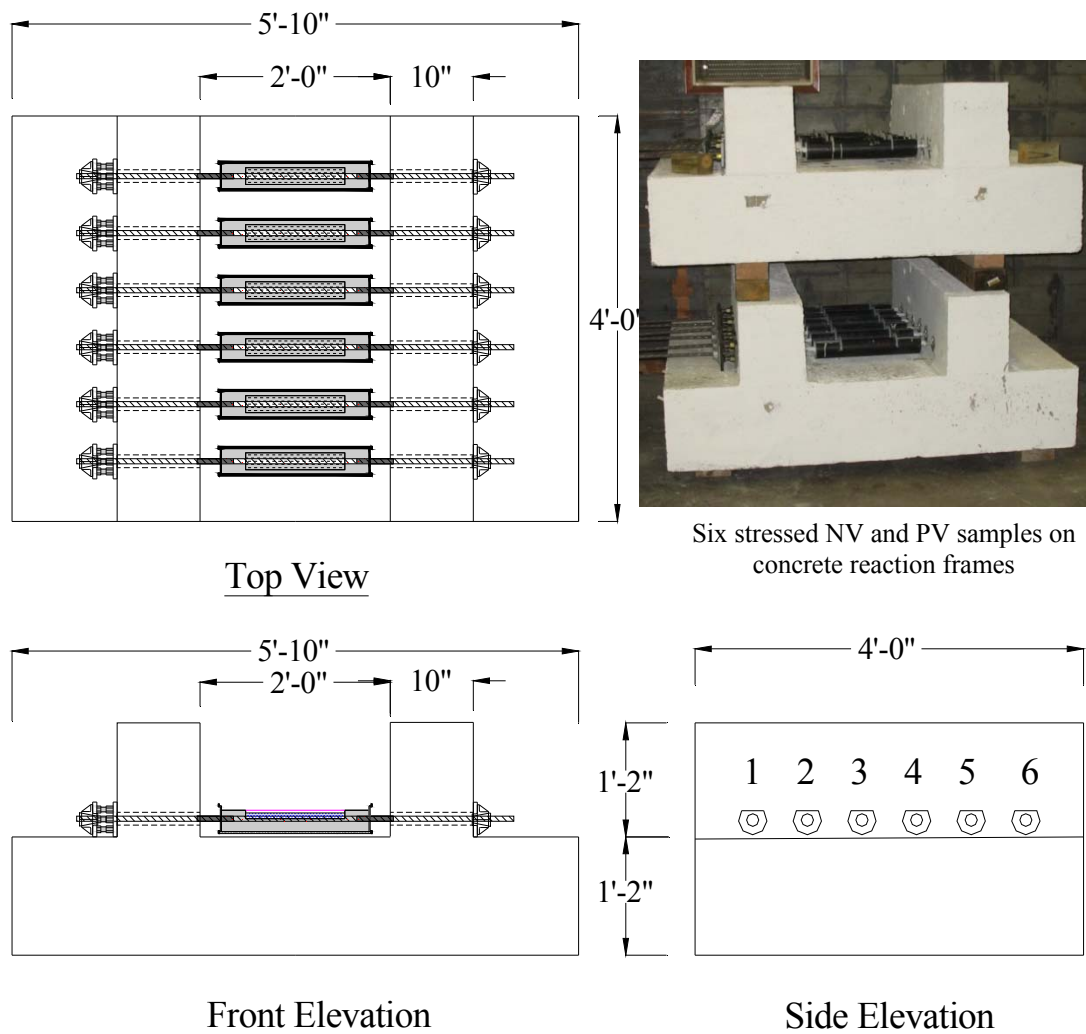
In the case of stressed strand corrosion testing, f_{axial} of approximately 150 ksi (1034 MPa) was applied and maintained by anchoring the strands to a specially designed concrete reaction frame (CRF). These CRFs were designed according to ACI 318-02/318R-02, *Building Code Requirements for Structural Concrete and Commentary*, to safely carry six stressed strand specimens. Structural detailing of the CRFs is provided in Appendix B. Twenty-seven CRFs with 6×4×3 ft (1.8×1.2×0.9 m) overall dimensions were made to expose and test 162 stressed strand specimens. Six, 10, and 11 CRFs were used to expose and test 36 NV, 60 PV, and 66 OV samples, respectively. Figure 5-11 shows the schematic and photograph of a vertical CRF exposing six OV specimens (indicated by the numbers in the top view diagram). Figure 5-12 shows the schematic of a horizontal CRF exposing six NV specimens (indicated by the numbers in the side elevation view diagram). The CRF setup for PV specimens is similar to that for NV specimens. The photograph on the top right corner of Figure 5-12 shows two horizontal CRFs exposing six NV and six PV specimens (stacked one on top of the other). The stressing operations are discussed next.



Six stressed OV samples on a concrete reaction frame

Note: All dimensions are in inches. 1 inch = 2.54 mm.

Figure 5-11. Schematic and Photograph of Vertical Concrete Reaction Frame with Stressed Strand Specimens.



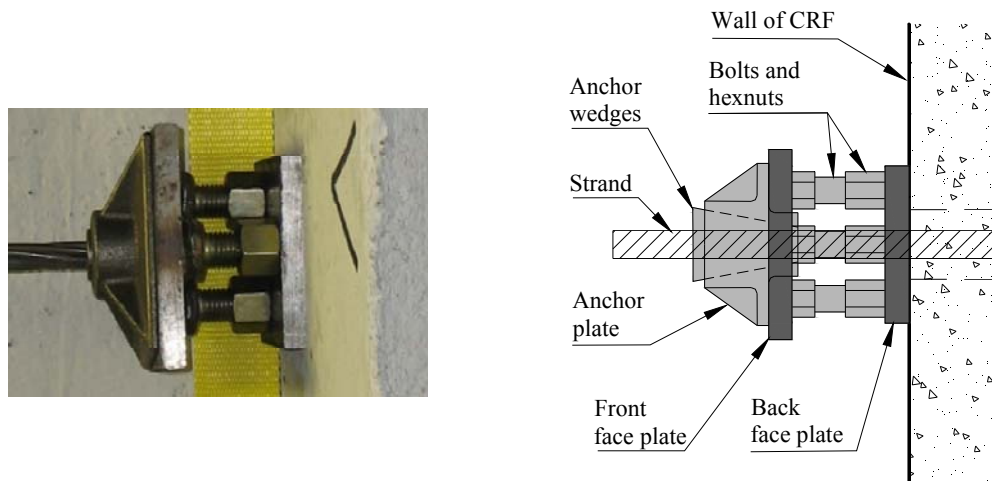
Note: All dimensions are in inches. 1 inch = 2.54 mm.

Figure 5-12. Schematic and Photograph of Horizontal Concrete Reaction Frames with Stressed Strand Specimens.

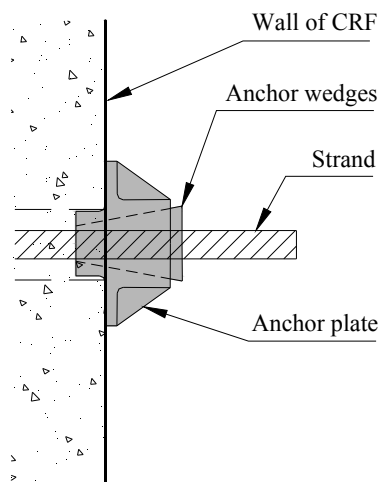
5.5.3.2. *Stressing systems*

The stressing system consisted of a monoram with a calibrated pressure gauge, a stress-adjusting system (defined later), an LVDT (attached to the strand being stressed), a computerized data acquisition system (to dynamically read the LVDT data), and a spreadsheet program (to calculate the required jacking and remaining axial stresses). The strand end at which the monoram was attached during stressing is known as the live end, and the other end is known as the dead end. In Figure 5-11, the live and dead ends of each stressed strand are shown at the top and bottom

regions, respectively. In Figure 5-12, the live and dead ends of each stressed strand are shown at left and right sides, respectively. Figure 5-13 shows more detailed schematics and photographs of the live and dead ends.



(a) Live end of stressing (shown at top and left side in Figure 5-11 and Figure 5-12, respectively)



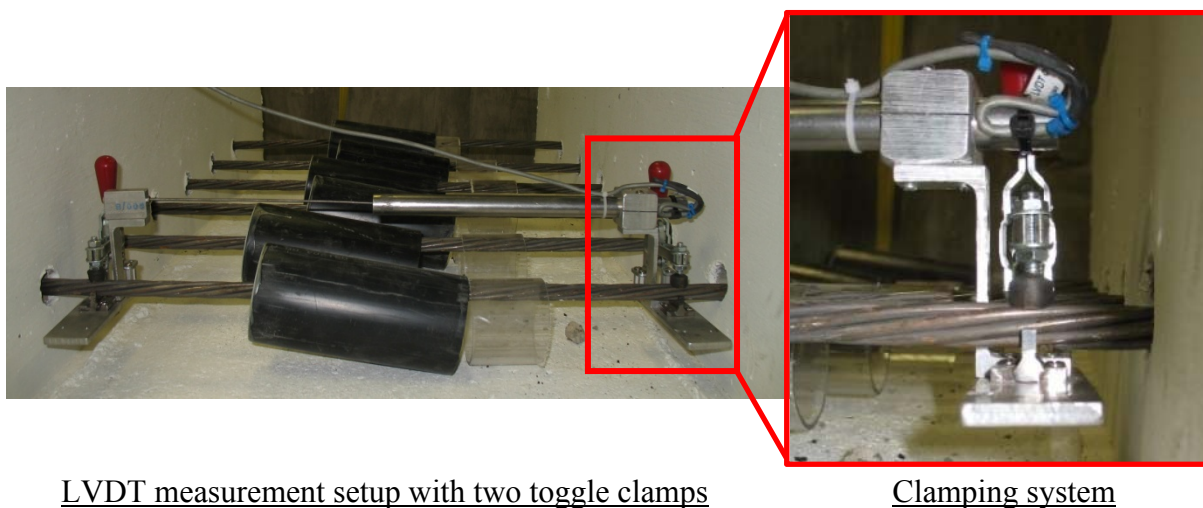
(b) Dead end of stressing (shown at bottom and right side in Figure 5-11 and Figure 5-12, respectively)

Figure 5-13. Schematics and Photographs of (a) Live End and (b) Dead End of the Stressed Strand on a Concrete Reaction Frame.

The system at live end, as shown in Figure 5-13(a), is defined here as a “stress-adjusting system,” which consists of three 0.75-inch (19 mm) diameter and 1.5-inch (38 mm) long bolts,

corresponding hexnuts, and two face plates. The front face plate was 5×5×0.5 inch (127×127×13 mm) in size. The back face plate was 4×4×0.5 inch (102×102×13 mm) in size. Both face plates have a center hole through which the strand can pass. The head of the three bolts on the system is welded to the front face plate on which it is resting. The three hexnuts rest on the bottom face plate. The dead end consists of a standard system with one anchor plate and two wedges (see Figure 5-13[b]).

During the stressing operations, the shortening of the strand was measured using the LVDT setup shown in Figure 5-14. A 10-inch (250 mm) long and 0.812-inch (21 mm) diameter LVDT (Schaevitz[®] Model 2000 HR) with a nominal linear travel range of ±2 inches (±51 mm) was used. The LVDT was connected to the strand using a special clamping system designed and fabricated in the laboratory. The clamping system consisted of two toggle clamps with neoprene pads and a knife edge (see right-hand figure in Figure 5-14). The body of the LVDT was connected to the right toggle clamp. The core of the LVDT was connected to the left toggle clamp using a 10-inch (250 mm) long, steel all-thread. The distance between the two knife edges was 22 inches (560 mm). This is because the inside-to-inside distance between the walls of the CRF was 24 inches (610 mm). The lead wire of the LVDT was connected to the laptop computer with a LabView[®] data acquisition program. The measured shortening was used to calculate the stress loss.



LVDT measurement setup with two toggle clamps

Clamping system

Figure 5-14. LVDT Setup to Measure the Strand Shortening during Stressing Operation.

A spreadsheet program was prepared to calculate the stress loss using the AASHTO (2004) formulations. This was also used to calculate the required jacking stress for repeated stressing operations of each strand (as discussed later). The strands were six sequentially stressed strands in the CRF.

5.5.3.3. *Stressing operations*

A sequential stressing operation was adopted to stress the six strands in each CRF. The sequence (or order) of stressing was selected in such a way that the possible unbalanced forces were minimized. The six strands on each CRF are indicated in the top and side elevation views in Figure 5-11 and Figure 5-12. On each CRF, the third strand was stressed first. The fourth, second, fifth, first, and sixth strands were then sequentially stressed. Each strand was repeatedly stressed until the required level of f_{axial} was attained. Figure 5-15 shows the schematic and photograph of the live end of the stressing setup in detail.

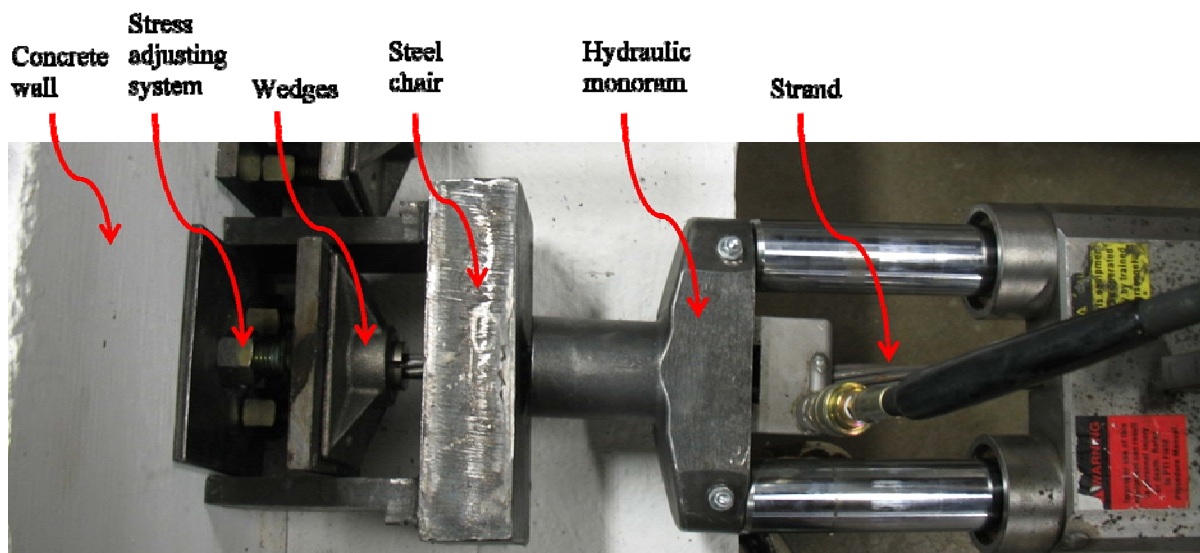
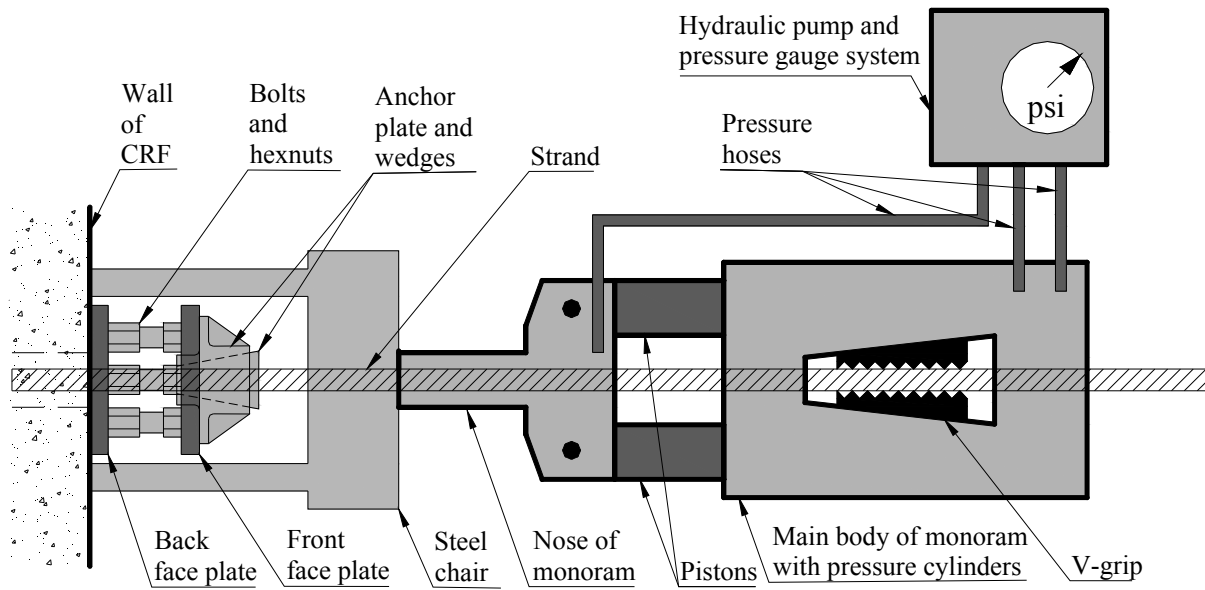


Figure 5-15. Schematic and Photograph of Strand Stressing Setup.

The strand was initially kept in place with the CRF. Then the “stress-adjusting system” was kept on the strand and the steel chair was kept in between the monoram and the wall of the CRF, as shown in Figure 5-15. Then the V-grip on the monoram was manually tightened. The hydraulic pump was then switched on, and the required jacking stress (calculated using the spreadsheet program) was applied. Then, as the monoram was resting on the steel chair, the three hexnuts were adjusted using an open-ended wrench. (To facilitate manual adjusting of the

hexnuts, there had to be no stress acting on them during the jacking operation. This was achieved by bypassing the jacking forces from the monoram to the strand through the steel chair. In other words, there was no stress acting on the anchor plates or wedges at the time of jacking. The stress was transferred to the anchor plates and wedges only after releasing the monoram.) The pump was then released and the strand shortened because of the additional seating of the anchor wedges and compression of the stress-adjusting system, crushing of concrete, etc. The overall strand shortening was measured using an LVDT and the data acquisition system. The measured shortening was recorded, and the stress loss was calculated using a spreadsheet program, which also calculated the additional required jacking stress. Then the same strand was stressed again using the same procedure discussed above. This process was repeated until sufficient stress was attained on the strand. Based on stress loss formulation in AASHTO (2004), the average stress on each strand during the exposure periods of 12 and 21 months were estimated to be 150.4 ksi (1037 MPa) with a standard deviation of 2.6 ksi (17.9 MPa).

5.5.4. Casting, curing, and exposure procedures

5.5.4.1. Strand specimens

Figure 5-9, shown earlier, provides the schematic of the strand specimens. All the strand specimens were prepared using 41-inch (1041 mm) long ASTM A416 strand pieces obtained from the same spool, heat, and lot. The 3-inch (76 mm) diameter HDPE pipe molds were prepared to facilitate casting of the grout for the strand corrosion specimens as follows. Schematics of the NV and PV samples are shown in Figure 5-9(a) and (b). HDPE pipes that were 18 inches (457 mm) long and capped at both ends were used to cast these samples. The capped pipes were cut along the length such that a 2×18 inch (51×360 mm) opening was formed. A 0.75-inch (19 mm) diameter hole was then drilled through the end caps. The strands were then placed through these end holes. The joints on the mold were sealed using waterproof silicone. Schematic of an OV sample is shown in Figure 5-9(c). HDPE pipes that were 7 inches (178 mm) long and capped at one end were used to cast these samples. A 0.75-inch (19 mm) diameter hole was then drilled on caps at one end of the pipes. The strands were then inserted through this hole. All the joints were sealed using waterproof silicone. Schematics of the IV and

BV samples are shown in Figure 5-9(d) and (e). To cast these samples, HDPE molds were prepared in the same way as was done for the OV samples, except that one end of the pipe was cut at a 45° angle. For all the specimens, the strands were placed and positioned inside the molds such that the NV, PV, OV, IV, and BV were formed after grouting. The strands were also fastened to the mold (using adhesive tape, as needed) to avoid movement while grouting. In the case of the unstressed strand specimens, temporary wooden frames were prepared to support the strands and molds while casting the specimen. In the case of stressed strand specimens, the strands and molds were self-supported by the CRFs. Freshly prepared grout was then placed in the space between the strand and HDPE molds (see Figure 5-9). Note that, in the case of the stressed strands, grouting was done after finishing the stressing operations.

These unstressed strand specimens were then cured (immediately after initial setting) at 73±3°F (23±1.5°C) and 95±5 %RH conditions for 28 days. The stressed strand specimens were cured at laboratory conditions at approximately 70–85°F (21–29°C) and 50–70 %RH for 28 days. After 28 days, each strand specimen was exposed to a pre-defined *MC* and %*sCl*⁻ level (see Subsection 5.5.2) using WD exposure to solutions with various chloride concentrations. Each WD cycle consisted of a two-week wet (or ponding) period followed by a two-week dry period. In Figure 5-9, the reservoir and ponding solutions are indicated by letters “R” and “P.” At the end of the exposure period, the stressed strand specimens were immediately removed from the CRFs using a cutting torch. The grout materials covering the unstressed and stressed strands were also immediately removed and the strands were visually assessed for corrosion characteristics and tested for tension capacity.

5.5.4.2. Wire specimens

Figure 5-10 shows the schematic and photograph of the wire corrosion test specimen (exposed to an OV condition). Each wire specimen was prepared using a 7-inch (178 mm) long king-wire that was obtained from the ASTM A416 strand. The standard plastic, self-standing, 1.5-inch (38 mm) diameter centrifuge tube with a conical bottom was used as the mold to cast the wire corrosion test specimen. The conical bottom facilitated the positioning of the wire at the center of the cylindrical grout covering. The cast specimens were then exposed to WD or CA

conditions. The experimental designs were shown and discussed earlier in Table 5-12 and Table 5-13.

For the WD conditions, the $\%sCT^-$ was attained by exposing the specimens to a WD environment using 0.0001, 0.006, 0.018, 0.18, and 1.8 $\%sCT^-$ solutions. A schematic of the test setup for the WD exposure is shown in Figure 5-16.

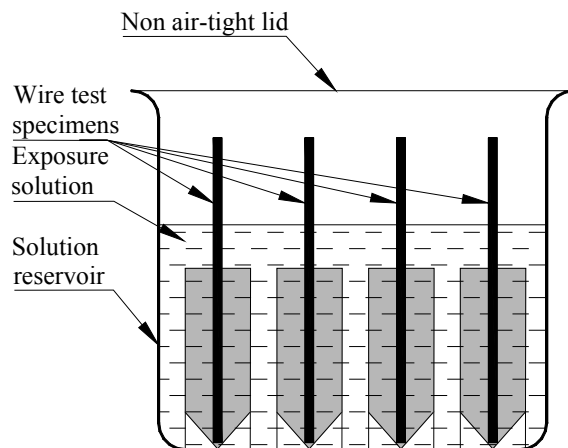


Figure 5-16. Exposure of Wire Specimens to WD Conditions.

For the wire specimens tested under CA exposure conditions, GAS interfaces with different chloride levels were obtained by immersing the specimens into 0.0001, 0.006, 0.018, 0.18, and 1.8 $\%sCT^-$ solutions. The specimens were immersed in the solutions one day after casting and continued for three days. Using the test procedure in SHRP (1992), the representative $\%gCT^-$ at the GAS interface of these test specimens were determined to be 0.0001, 0.014, 0.092, 0.343, 0.782, respectively. The wire specimens with different $\%gCT^-$ levels were then placed in controlled environments with pre-defined levels of RH and T , as shown in Table 5-13. The RH and T were controlled as discussed in the following paragraphs.

To maintain a 100 $\%RH$, the ASTM D5032, *Standard Practice for Maintaining Constant Relative Humidity by Means of Glycerine Solutions*, was followed. According to this, a 0 percent glycerine solution inside an air-tight container will maintain 100 $\%RH$ for an air-column of 10 inch³ (16.4 cm³) or less per inch² (per cm²) of water surface. Following this approach, the wire specimens were kept inside a 6 × 6 × 10 inch (150 × 150 × 250 mm) air-tight plastic

container, as shown in Figure 5-17. The plastic molds used to cast the wire specimens were not removed, and the water level inside the containers was kept below the GAS interface. This was done to keep the wire or the GAS interface region free from direct contact with water. Note that this arrangement does not prevent the potential formation of H₂O molecules (due to condensation) at the GAS interface region or on the wire surface. Three such setups, as shown in Figure 5-17, were prepared and were kept in walk-in environmental rooms with T of 43, 85, and 94°F (6.1, 29.4, and 34.4°C), respectively.

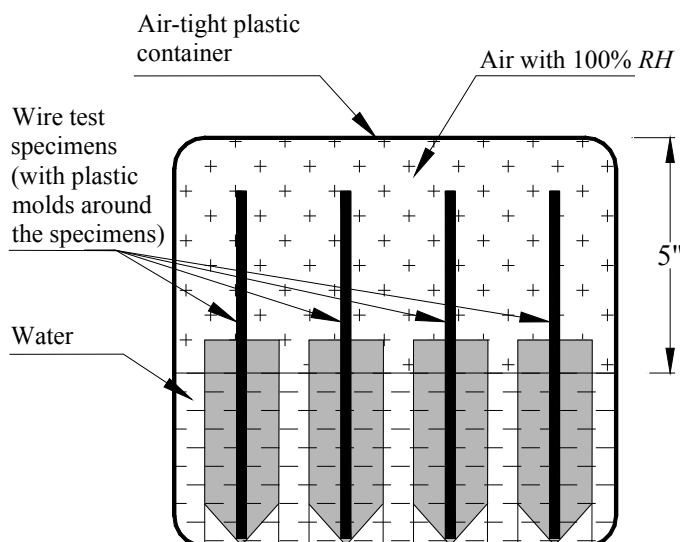
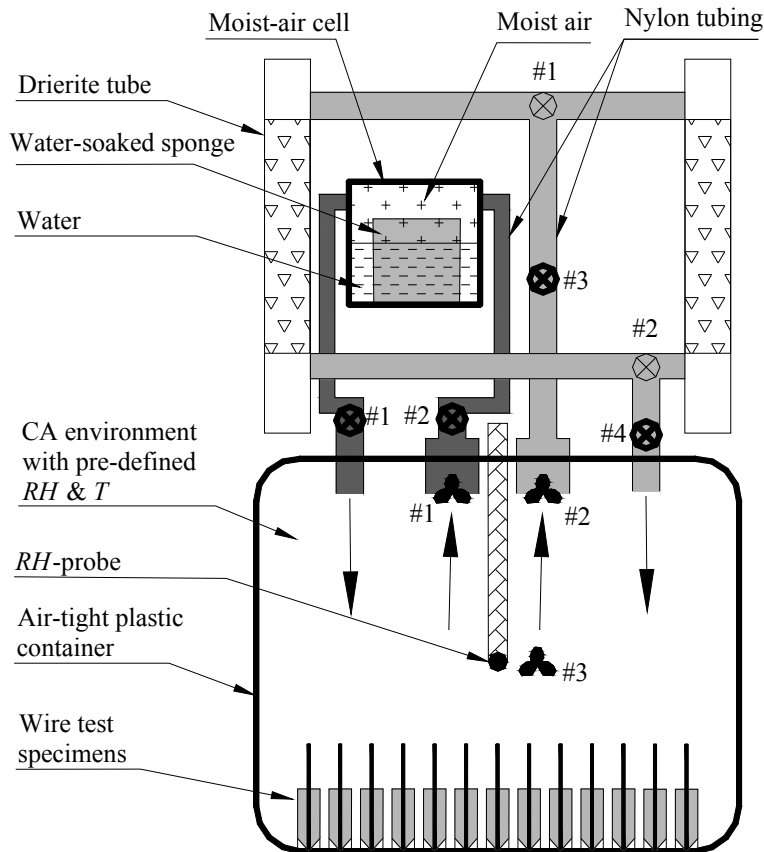


Figure 5-17. Exposure of Wire Specimens to CA Conditions with 100 %RH.

To maintain a 45 or 70 %RH, the test setups as shown in Figure 5-18 were developed. In this setup, the wire specimens were kept inside a 32 × 19 × 16 inch (8128 × 483 × 406 mm) air-tight plastic container. The volume of this container was 31 gallons (117 liters). This much air-volume was necessary to effectively control the RH, using the “RH-control system” discussed later. Three such air-tight containers were fabricated and were kept inside three walk-in environmental rooms with a T of 43, 85, and 94°F (6.1, 29.4, and 34.4°C), respectively. The RH inside this container was controlled using an “RH-control system” consisting of two Drierite[®] columns, one container with a water-soaked sponge, four solenoid valves, two manual ball

valves, and two mini-fans. These components were connected using clear, flexible nylon tubing (shown in light and dark grey colors in Figure 5-18) to carry air to and from the air-tight container. The solenoid valves and mini-fans were connected to a dual-channel electronic controller (shown in the photograph; not shown in the schematic). The first channel of the controller was set such that when the *RH* was two percent less than the pre-defined level, the solenoid valves #1 and #2 would open and the mini-fan #1 would switch on, resulting in a flow of air through the “moist-air cell.” The sponge inside the moist-air cell was soaked with water. Because of this, the water-body attained an effectively larger surface area and more H₂O molecules could continuously diffuse into the atmosphere inside the moist-air cell. Therefore, the dry-air flowing into the moist-air cell became moist. The moistened air then flowed back into the air-tight container with the wire specimens, resulting in an increase in *RH*. The second channel of the electronic controller was also set such that when *RH* was two percent greater than the pre-defined level, the solenoid valves #3 and #4 would open and the mini-fan #2 would switch on, resulting in a flow of air through the Drierite[®] tube. The dried air flowed back into the air-tight container, resulting in a reduction in *RH*. Mini-fan #3 was run continuously to facilitate good air circulation in the container. During the preliminary development of the setup, it was found that Drierite[®] becomes saturated with moisture within a week. Because of this, a second Drierite tube (as a back-up) was installed on each setup. Only one Drierite tube was used (selected using the manual ball valve) at a time, while the other was being de-saturated or dried.



Notes:

The mini-fans, solenoid valves, and RH-probe are connected to the electronic controller system, which is not shown in the schematic.





-  Mini-fan
 -  Manual ball valve
 -  Solenoid valve
-  Indicates the direction of air-flow

Figure 5-18. Exposure of Wire Specimens to CA Conditions with 45 and 70 %RH.

As discussed, all the wire specimens were exposed to WD and CA exposure conditions. At the end of the pre-defined exposure period (see Table 5-12 and Table 5-13), the grout material on the wire specimens was removed. The exposed wire specimens were then inspected for corrosion characteristics, as discussed next.

5.5.5. Corrosion evaluation procedures

Subsection 2.4 provided a review of existing methods to clean and inspect strands in the field. In the current research program, wires and strands were hand-rubbed with a synthetic cleaning pad until the loose corrosion products or rust was removed. In addition, small steel wire brushes were used to remove harder corrosion products that were difficult to remove with the synthetic cleaning pad. Photographs and micrographs of the GAS interfaces on strand and wire specimens were then collected. These photographs were then used to characterize surface corrosion, pitting corrosion, corrosion at or near GAS interfaces, or other unique patterns that could be found on corroding PT strands. These photographs of corroded PT strands (taken after cleaning) were compared with the corresponding tension capacity and can be used as a visual tool to estimate the tension capacity of strands in actual PT structures.

5.5.6. Tension capacity determination or estimation procedures

Tension capacities of the strand and wire specimens were determined using separate procedures. These procedures are discussed next.

5.5.6.1. Strand specimens

The remaining tension capacity of the corroded strands were determined using ASTM A 370-02, Standard Test Methods and Definitions for Mechanical Testing of Steel Products, and ASTM E 111-02, Standard Test Method for Young's Modulus, Tangent Modulus, and Chord Modulus. A 400-kip (1780 kN) universal testing machine was used for the testing and was equipped with steel hydraulic jaws and aluminum plates to grip the strand specimens at each end. Each aluminum plate (made of Alloy 2024 meeting ASTM B 211-03, *Standard Specification for Aluminum and Aluminum-Alloy Bar, Rod, and Wire*, requirements and with a T351 temper) was 0.75 inches (19 mm) thick, 2 inches (50 mm) wide, and 7 inches (178 mm) long and had a 7-inch (178 mm) long semi-circular groove with a 0.25-inch (6 mm) radius. Prior to testing, a 7-inch (178 mm) long "as-received" strand was placed along the groove and sandwiched between the two plates and then compressed to 250 kips (1112 kN) such that spiral indentations were formed. These grooved surfaces allowed for better seating of the strand specimen and reduced slip. Also,

a thick paste made of aluminum oxide and glycerin was applied inside the grooves to improve the grip. This gripping setup also helped to minimize the stress concentration at the grips. The elongation over the 24-inch (610 mm) gage length between the two grips was measured using a LVDT (with ± 2 inches (± 51 mm) nominal travel range) fastened to the strand using a knife-edge holder.

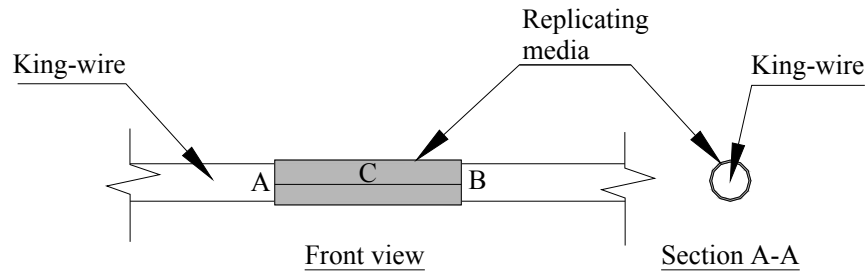
5.5.6.2. Wire specimens

Tension capacities of 7-inch (178 mm) long wire specimens exposed to WD conditions were evaluated using a 100-kip (445 kN) tension testing machine and following ASTM A 370-02 and ASTM E 111-02 standard test procedures.

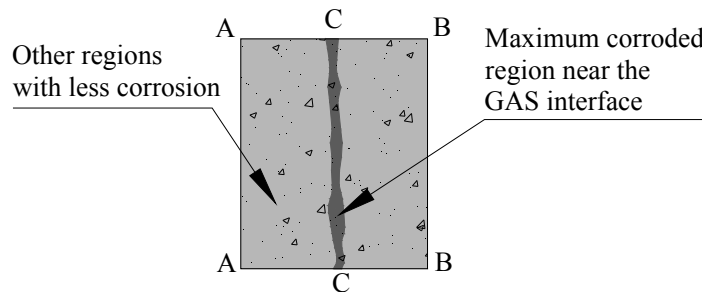
Among the 61 wire samples exposed to the various CA conditions for a period of 9 months, tension capacity of 15 samples was determined using a 100-kip (445 kN) tension testing machine and ASTM A 370-02 and ASTM E 111-02 standard test procedures. Some of these wire capacity values were statistically similar to the pristine wire capacity. However, localized corrosion was visible at GAS interfaces of these specimens, and they failed in tension or broke at the GAS interface region. This indicates that the weakest cross-section was at or near the GAS interface region and there was a very small reduction in tension capacity, which was not sufficient or sensitive enough for the 100-kip (445 kN) tension testing equipment and standardized laboratory testing procedures to reliably determine. However, it was necessary to estimate this small reduction in tension capacity to develop the probabilistic models in Section 9.1. An indirect or novel approach is developed to calculate the tension capacity of unstressed wires, $C_{T,UW}$, using the corresponding cross-sectional area loss of unstressed wire samples, $A_{loss, UW}$.

$A_{loss, UW}$ was determined using a Dektak[®] 3 Stylus profilometer, which can measure vertical profiles ranging in height from 3.94×10^{-7} to 2.56×10^{-3} inches (10 to 65,000 nm) and has a stylus tracking force equal to 1.1×10^{-4} lb (4.9×10^{-4} N). At first, the wire specimens were cleaned using the synthetic pad, as described in Subsection 5.5.5. Figure 5-19 depicts the procedure to calculate cross-sectional area using surface replicating media and a profilometer. Figure 5-19(a), (b), and (c) are not drawn to same scale and, hence, the letters “A,” “B,” and “C”

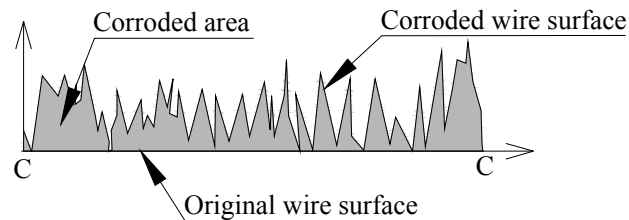
are used as reference points. Then, as shown in Figure 5-19(a), the GAS interface region on the wire specimen was covered with a surface replicating media (Model No. 57-6002, manufactured by Beuhler Ltd.). The cured replicating media was cut through the line segment \overline{AB} on Figure 5-19(a) and removed from the wire specimen. Figure 5-19(b) shows the inside face of the cured replicating media, which was in contact with the wire surface. The surface profile along the darker center-region in Figure 5-19(b) represents the profile of the GAS interface region and was then obtained using Dektak[®] 3 Stylus profilometer. Figure 5-19(c) shows a typical surface profile obtained using this technique. The shaded region in Figure 5-19(c) indicates $A_{loss, UW}$ and was calculated using the Dektak[®] 3 software.



a) Replicating media on GAS interface region on a wire specimen



b) Typical inside face of a cut-open replicating media



c) Typical profile obtained using Dektak 3 Stylus profilometer

Note: All drawings are not on same scale.

Figure 5-19. Cross-Sectional Area Measurement Using Surface Replicating Media and Profilometer.

A graphical representation of the analytical procedure to determine $C_{T,UW}$ at 9 months is provided in Figure 5-20. Note that Figure 5-20 is not drawn to scale.

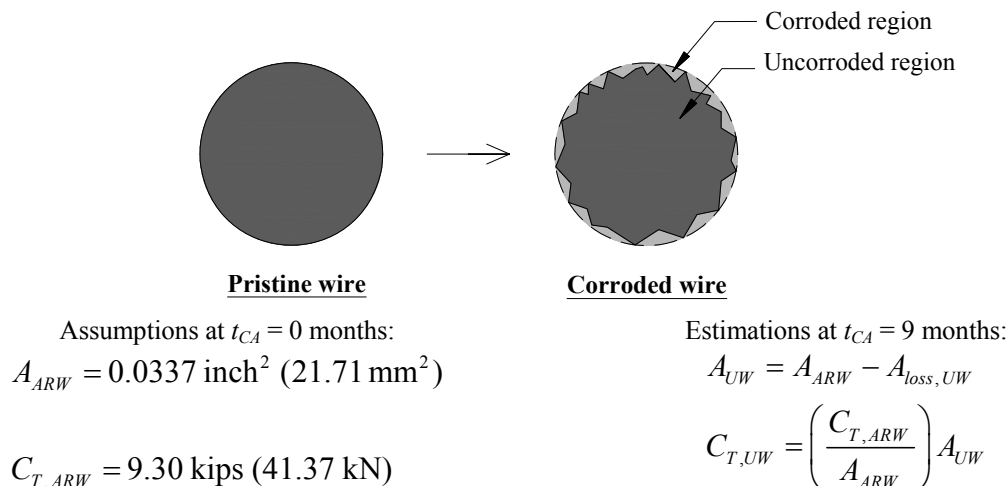


Figure 5-20. A Graphical Representation of the Analytical Determination of Tension Capacity of Wire Specimens under 9-Month CA Exposure.

The detailed procedures for the analytical determination of $C_{T,UW}$ at 9 months using $A_{loss,UW}$ at 9 months are as follows:

1. The capacity of pristine wire, $C_{T,UW}$, is defined and assumed as follows:

$$C_{T,ARW} \Big|_{max} = Max(C_{T, all ARW}) = 9.30 \text{ kips (41.37 kN)} \quad (5.3)$$

Due to material heterogeneity, measurement error, and other reasons, there exists a small scatter (ranging approximately 0.2 kips) among the pristine wire capacity values observed. Moreover, the cross-sectional area loss at t_{CA} equal to 9 months, $A_{loss,UW}$ at 9 months, was very small. In order to numerically express the corresponding small capacity loss in 9 months, a deterministic value for $C_{T,ARW}$ needs to be assumed. Also, in order to avoid the mathematical problems of obtaining a negative capacity loss, $C_{T,ARW}$ has to be set larger than or equal to all the experimentally observed wire capacities. In all the tension testing of wires, the largest capacity observed was 9.3 kips. Hence, $C_{T,ARW}$ is assumed to be 9.3 kips.

2. The cross-sectional area of pristine wire, A_{ARW} , is assumed as follows:

$$A_{ARW} = \frac{\pi d_{ARW}^2}{4} = \frac{3.1415 \times 0.207^2}{4} = 0.0337 \text{ inch}^2 (21.71 \text{ mm}^2) \quad (5.4)$$

where d_{ARW} = diameter of pristine wire.

3. Determine $A_{loss,UW}$ at t_{CA} equal to 9 months using surface profilometry techniques given in Subsection 5.5.6.
4. Determine the available cross-sectional area of wires, A_{UW} , at t_{CA} equal to 9 months as follows:

$$A_{UW} = A_{ARW} - A_{loss,UW} \quad (5.5)$$

5. Estimate the corresponding wire capacity loss, $C_{T,UW}$, at t_{CA} equal to 9 months as follows:

$$C_{T,UW} = \left(\frac{A_{ARW}}{C_{T,ARW}} \right) A_{UW} \quad (5.6)$$

Now, the superset of capacity data for wire specimens under CA exposure conditions can be developed by combining the capacity data obtained from both tension testing (15 wire specimens) and surface profilometry/analytical procedures (46 wire specimens). Although some of these data are analytically estimated or derived, this wire capacity will be referred to as “observed” or “measured” wire capacity herein. The full data set is provided in APPENDIX A.

5.6. SUMMARY

This section presented the details of the experimental program. At first, the characteristics of the materials used in this experimental program were discussed. Then the objectives, design, layout, and procedures of various test programs were discussed. The test programs presented were cyclic polarization, galvanic corrosion, and strand/wire corrosion tests.

6. ELECTROCHEMICAL CHARACTERISTICS AND TENSION CAPACITY OF ASTM-A416 STRANDS AND WIRES

Equation Section (Next)

6.1. INTRODUCTION

This section presents the electrochemical and tension capacity results from the experimental program presented in Section 4. The electrochemical characteristics obtained from the cyclic polarization tests are presented first. The galvanic corrosion characteristics obtained from the modified ASTM G109 and bearing plate tests are then presented. Following this, the corrosion patterns observed on the tension capacity, C_T , of strand and wire corrosion test specimens are presented. The critical parameters influencing C_T of strands are then identified.

6.2. CYCLIC POLARIZATION TEST RESULTS

Subsection 5.3 presented the experimental program and procedures on cyclic polarization tests. Subsection 5.3.1 defined some electrochemical terms used in cyclic polarization curves. Figure 6-1(a), (b), and (c) show the cyclic polarization curves for ASTM A416 steel specimen when immersed in simulated concrete pore solutions with hydroxides and 0.0, 0.06, and 1.8 %sCl⁻. The solid arrows along the curve indicate the direction of scan (i.e., forward and reverse scans). The horizontal dashed arrows indicate the electrochemical parameters (i.e., E_b , E_{rp} , and E_{pp}) defined in Subsection 5.3. Note that all the potentials discussed in this subsection are measured with reference to that of Saturated Calomel Electrode (E_{SCE}).

Cyclic polarization curves in Figure 6-1(a) and (b) show negative hysteresis, indicating that either the passive films formed on the steel surface are self-repaired or corrosion pits are not initiated when immersed in simulated pore solutions with 0.0 and 0.06 %sCl⁻. However, the curve in Figure 6-1(c) shows a positive hysteresis. This indicates that either the passive films are not repaired or corrosion pits initiate when the ASTM A416 steel is immersed in simulated pore solution with 1.8 %sCl⁻.

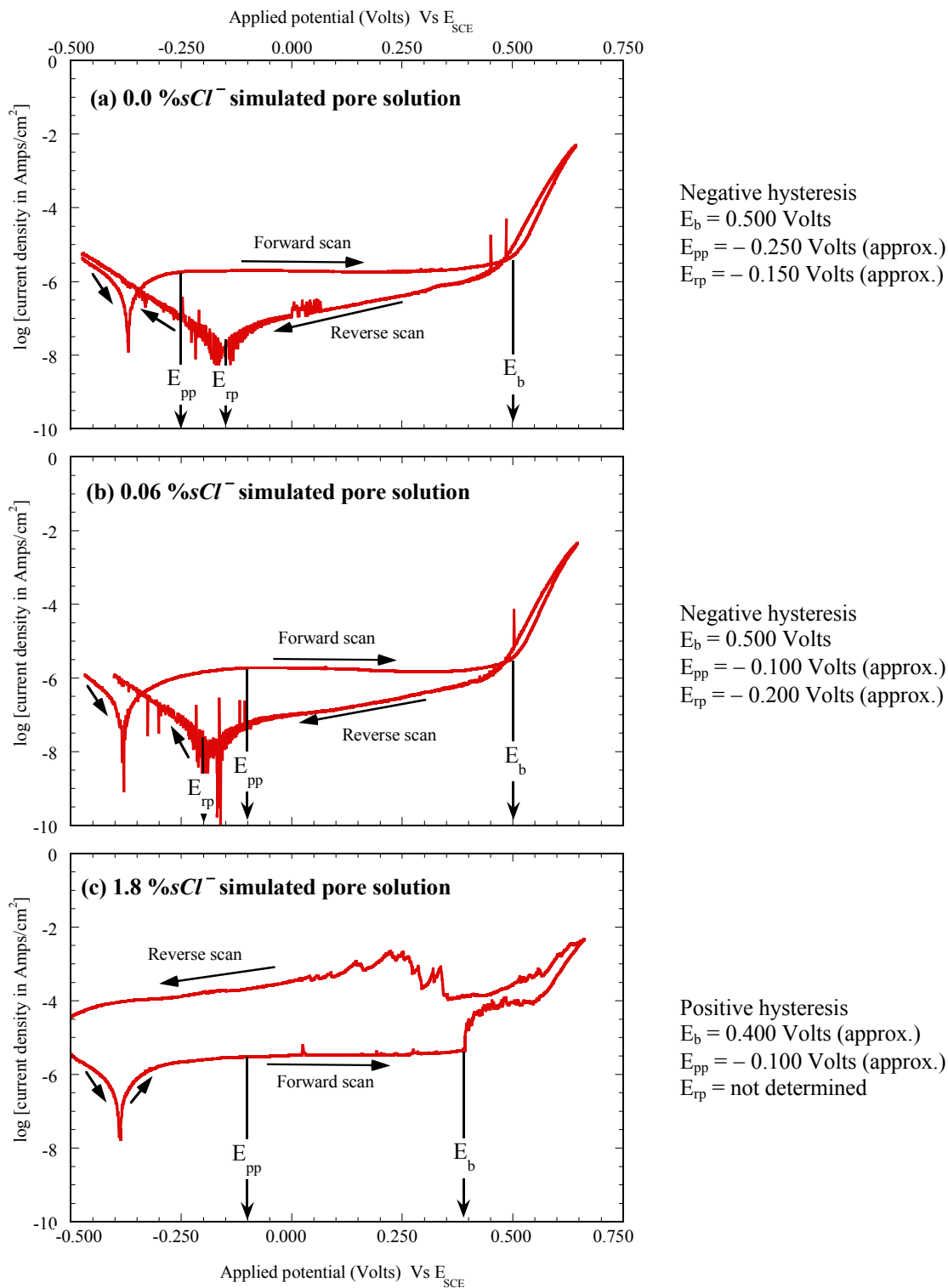


Figure 6-1. Cyclic Polarization Curves for ASTM A416 Steel.

The open circuit potential, OCP, of the ASTM A416 steel specimen when immersed for one hour in simulated pore solutions with 0.0, 0.06, and 1.8 %sCl⁻ were determined to be approximately -0.370, -0.380, and -0.390 Volts, respectively. This indicates that the OCP remained approximately constant as the %sCl⁻ increased from 0.0 to 0.06 and 1.8. When immersed in simulated pore solutions with 0.0, 0.06, and 1.8 %sCl⁻, the current measured corresponding to the open circuit conditions were 3.7×10^{-6} , 4.7×10^{-6} , and 8.4×10^{-6} Amps/inch² (0.57×10^{-6} , 0.73×10^{-6} , and 1.30×10^{-6} Amps/cm²), respectively. This indicates 1.3 and 2.3 times increase in the open circuit current as %sCl⁻ increases from 0.0 to 0.06 and 1.8, respectively.

The breakdown potential, E_b, in Figure 6-1(a) and (b) are 0.500 Volts and in Figure 6-1(c) is approximately 0.400 Volts. This indicates that the passive film on ASTM A416 steel breaks down at a lower potential when immersed in 1.8 %sCl⁻ solution than when immersed in 0.0 or 0.06 %sCl⁻ solutions, which would be expected.

When immersed in simulated pore solutions with 0.0 %sCl⁻, the primary passivation potential, E_{pp}, was approximately -0.250 Volts. E_{pp} was approximately -0.100 Volts when immersed in simulated pore solutions with 0.06 and 1.8 %sCl⁻. Although these values are approximate, they indicate that passive films on ASTM A416 steel immersed in simulated pore solutions with 0.06 and 1.8 %sCl⁻ are stable when the potential is between -0.100 Volts and the corresponding E_b. These values show that the window of the passive region decreases as %sCl⁻ increases. The variation in the environment in the field could induce a variation in the potential of PT strands, indicating more probability for active corrosion to occur with a narrower window of passive region.

The repassivation potential, E_{rp}, was determined to be -0.150 and -0.200 Volts when immersed in simulated pore solutions with 0.0 or 0.06 %sCl⁻. When immersed in the simulated pore solution with 1.8 %sCl⁻, a positive hysteresis was observed and repassivation did not occur during the cyclic polarization test. Because of this, E_{rp} could not be determined. However, it can be concluded that E_{rp} is more negative than -0.490 Volts, the potential at which the cyclic polarization test was started and ended.

Table 6-1 summarizes the approximate active, passive, and transpassive potential regions for the ASTM A416 steel. These potential regions are determined based on the forward scans in Figure 6-1(a), (b), and (c). In general, the passive region becomes narrower as %sCl⁻ increases.

Table 6-1. Approximate Active, Passive, and Transpassive Potential Regions for the ASTM A416 Steel.

Simulated pore solution with	Active region	Passive region	Transpassive region
0.0 %sCl ⁻	< -0.250 Volts	Between -0.250 and +0.500 Volts	> +0.500 Volts
0.06 %sCl ⁻	< -0.100 Volts	Between -0.100 and +0.500 Volts	> +0.500 Volts
1.8 %sCl ⁻	< -0.100 Volts	Between -0.100 and +0.400 Volts	> +0.400 Volts

6.3. GALVANIC CORROSION TEST RESULTS

This subsection presents the results obtained from the modified ASTM G109 and bearing plate tests presented in Subsection 5.4.

6.3.1. Modified ASTM G109 test results

Figure 6-2 and Figure 6-3 show the total corrosion observed on the modified ASTM G109 samples with WD exposure conditions to 0 and 9 %sCl⁻ solutions, respectively. In Figure 6-2 and Figure 6-3, the hollow and solid data markers indicate the samples cast with Class A and C grouts, respectively. In both Figure 6-2 and Figure 6-3, the samples with Class A and C grouts show similar corrosion behavior. Figure 6-2(a) shows the total corrosion on the samples with similar anode and cathode material (i.e., non-galvanic samples) and exposed to 0 %sCl⁻ solution. Figure 6-2(b) shows total corrosion on the samples with dissimilar anode and cathode material (i.e., galvanic samples) and exposed to 0 %sCl⁻ solution. The total corrosion in both Figure 6-2(a) and (b) is negligible and similar in nature. This indicates that the presence of moisture only cannot initiate galvanic corrosion between the ASTM A416 and A615 steel at the anchorage zones in PT systems. The absence of galvanic corrosion also indicates that the corrosion potentials of ASTM A615 and A416 steels are similar when no-chloride conditions exist.

Figure 6-3(a) and (b) show the total corrosion on the non-galvanic and galvanic samples, respectively, exposed to 9 %s Cl^- solution. In general, the total corrosion was similar in both non-galvanic and galvanic samples. However, some samples in Figure 6-3(b) show a slightly earlier onset of corrosion than those in Figure 6-3(a). Still, this difference is not sufficient to clearly state that galvanic corrosion occurred in the latter case. Further experiments with more parameter combinations is needed but not performed as part of this study.

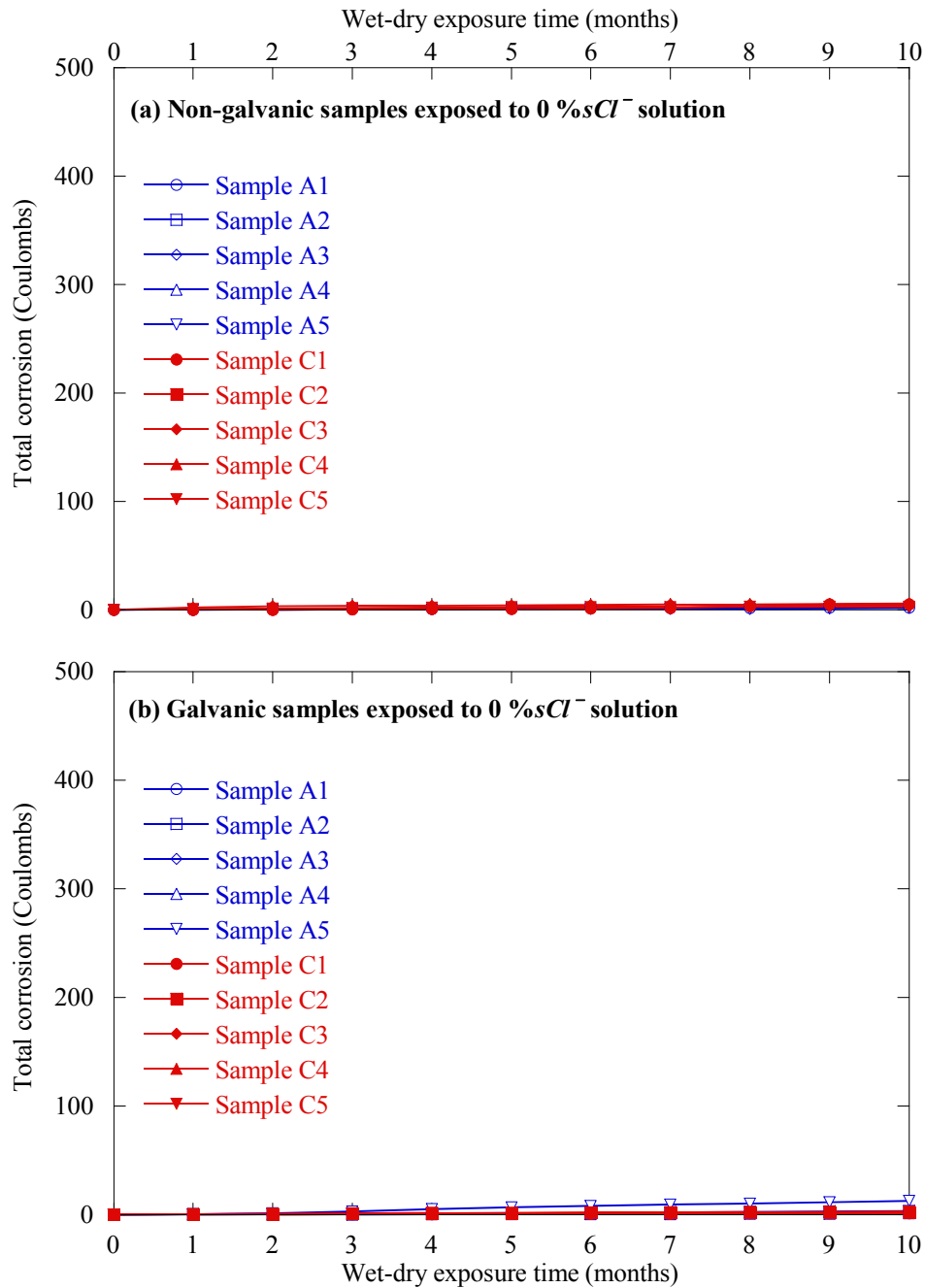


Figure 6-2. Total Corrosion on the Modified ASTM G109 Samples with Wet-Dry Exposure to 0 %sCl⁻ Solution.

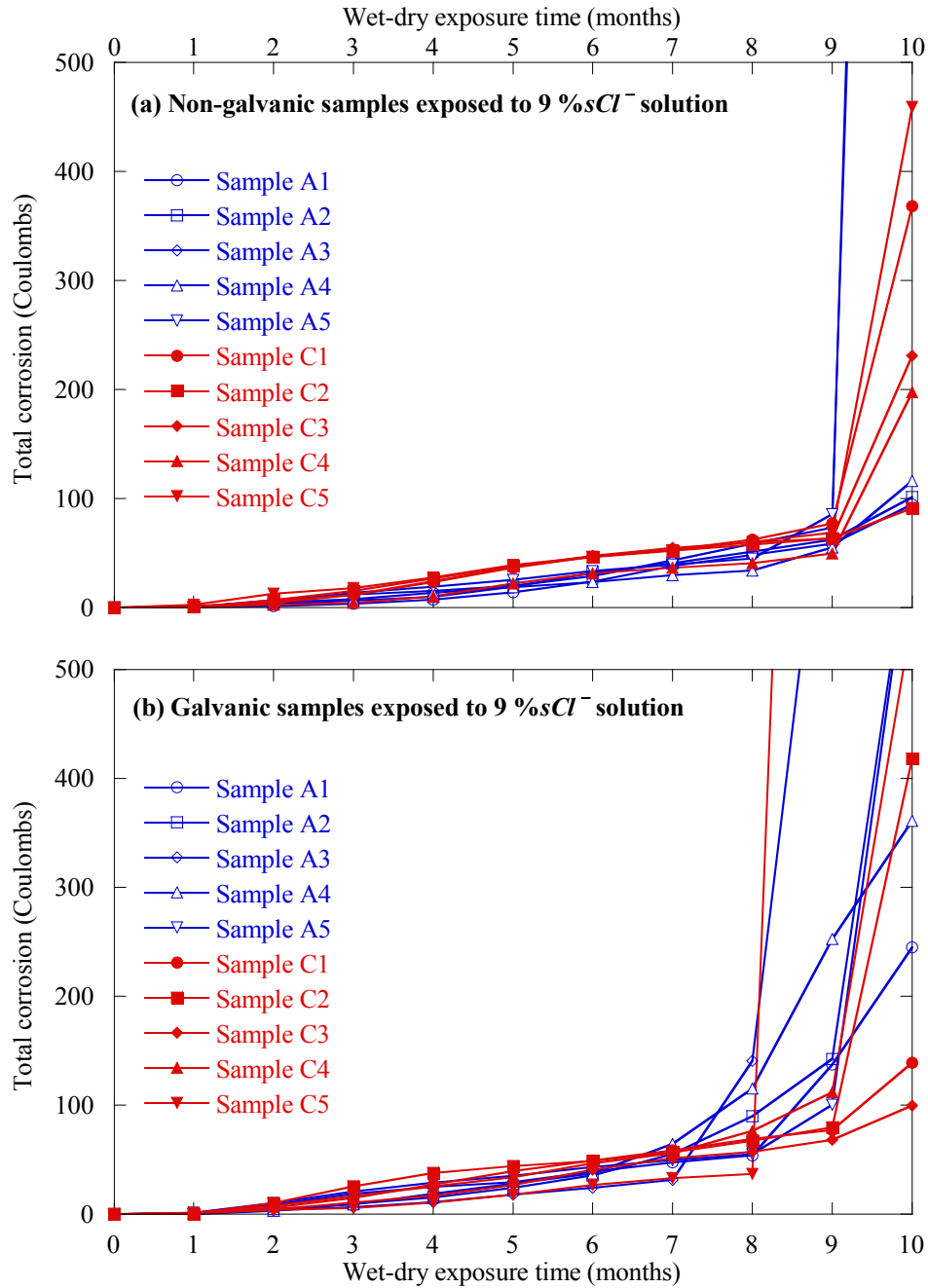


Figure 6-3. Total Corrosion on the Modified ASTM G109 Samples with Wet-Dry Exposure to 9 %sCl⁻ Solution.

6.3.2. Bearing plate test results

Subsection 2.3.1 indicated that cast iron is more noble than steel when immersed in seawater. Figure 6-4 and Figure 6-5 show the total corrosion observed on the bearing plate samples with WD exposure to 0 and 9 %sCl⁻ solutions, respectively. In both Figure 6-4 and Figure 6-5, active corrosion of the conventional spiral reinforcement initiated at approximately 9 months (or after 9 WD cycles). This indicates that the galvanic corrosion between ASTM A615 and A536 (cast iron) does occur and moisture alone is sufficient to initiate galvanic corrosion between ASTM A615 and A536 in PT systems.

The modified ASTM G109 tests indicated that the corrosion potentials of ASTM A615 and A416 steels are similar. Bearing plate tests indicated that galvanic corrosion can occur between ASTM A615 and A536 steels. Using these two results, it can be concluded that moisture alone is sufficient to initiate galvanic corrosion between ASTM A416 and A536 in PT systems. Also, the presence of high amounts of chlorides can accelerate corrosion between ASTM A416 and A536 in PT systems.

These test results indicate that the surface of the bearing plates may be coated with epoxy or other non-conductive material such that the galvanic corrosion between bearing plate and other metallic materials in the PT systems can be minimized.

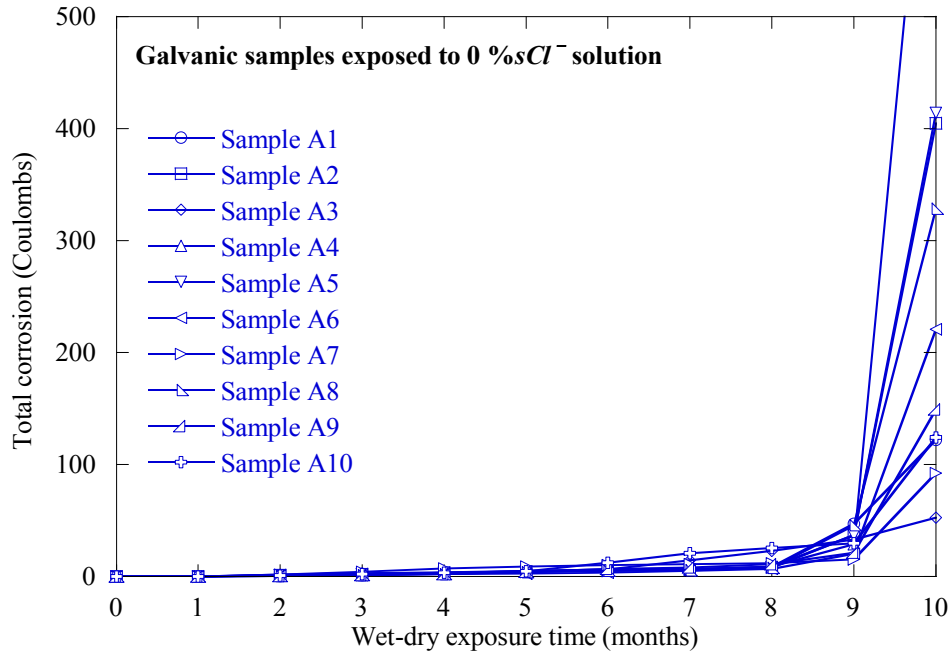


Figure 6-4. Total Corrosion on the Bearing Plate Samples (0 %sCl⁻ Solution).

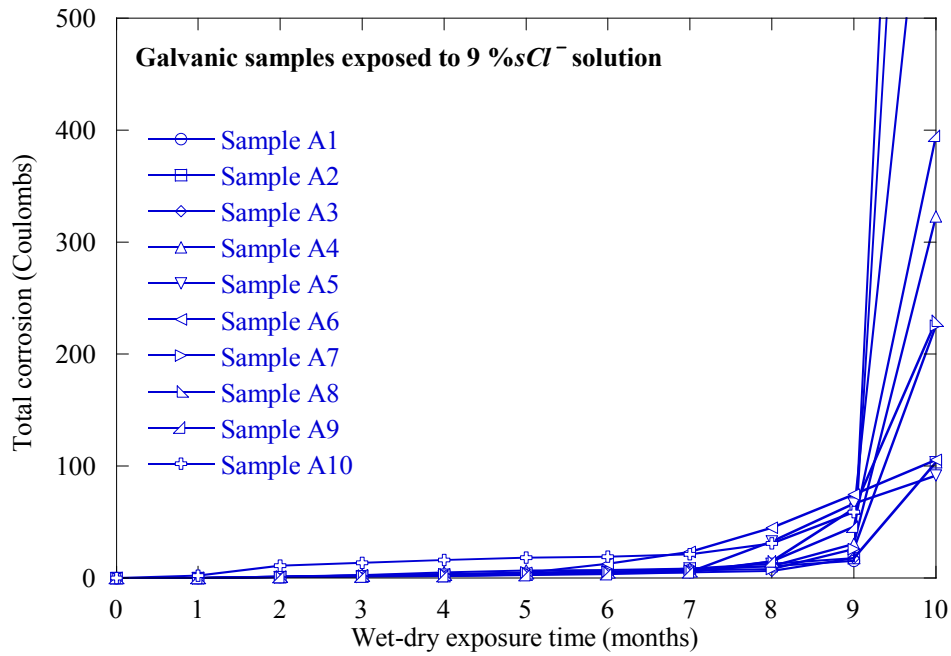


Figure 6-5. Total Corrosion on the Bearing Plate Samples (9 %sCl⁻ Solution).

6.4. STRAND AND WIRE CORROSION TEST RESULTS

This subsection presents experimental observations from the strand and wire corrosion test programs. The details of the experimental program were provided earlier in Subsection 5.5. A discussion on general observations of the cleaned strand and wire surfaces are provided first. Then a collection of photographs of corroded strands to estimate the C_T of strands is provided. Following this, dot plots showing C_T data generated from the strand and wire corrosion tests are provided.

6.4.1. General visual observations

PT strands are made of six outer wires spiraled around a straight center king-wire resulting in a flower-like cross-sectional geometry. This geometry includes interstitial spaces between the king wire and outer wires. These small interstitial spaces may initiate and propagate crevice and other accelerated forms of corrosion. However, the gradual build-up of corrosion products in the interstitial spaces could result in reduced oxygen availability at these spaces, thus slowing the corrosion process. These conditions could lead to a more severe corrosion at the outer wire surface than on the center king wire. Figure 6-6 shows micrographs of a strand exposed to wet-dry conditions with high % sCl^- , resulting in severe pitting corrosion. These pits are not clearly visible in Figure 6-6. However, they were visible through a microscope and were similar to the pits observed on reinforcement surfaces in chloride contaminated concrete systems. Most of this corrosion damage occurred near the GAS interface. In addition, the examination of the micrograph indicates linear patterns of corrosion along the longitudinal axis of the outer wires. This could be attributed to the surface treatment during the cold wire drawing or manufacturing process. The adverse effect of this maximum corrosion near the GAS interface was validated when the strands failed at the GAS interface during tension testing. Wire corrosion test specimens also exhibited a similar type of corrosion behavior near the GAS interface.

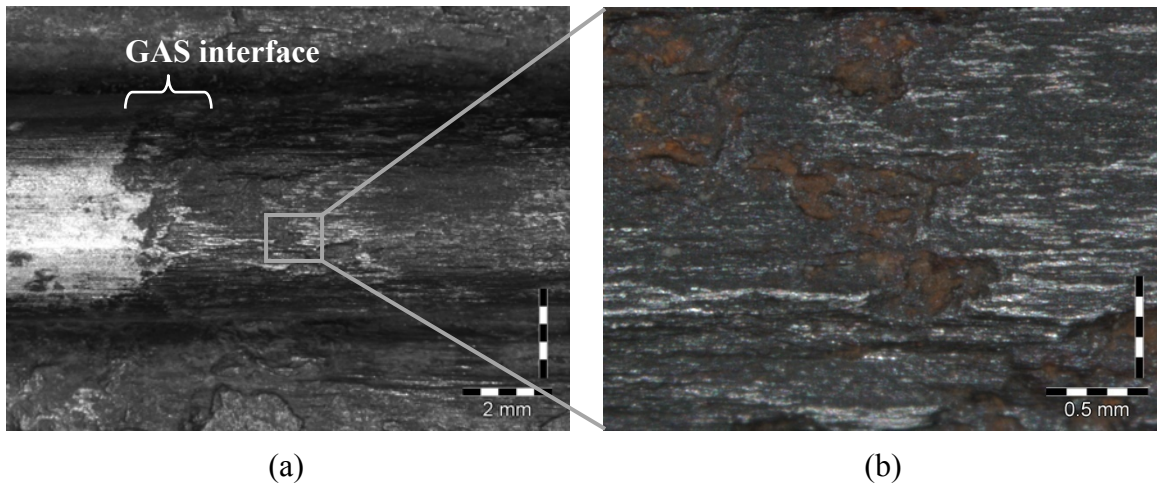


Figure 6-6. Micrographs Showing (a) Localized Corrosion near Grout-air-strand (GAS) Interface; (b) Linear and Pitting Patterns of Corrosion.

6.4.2. Estimating tension capacity from strand photographs

The first column in Figure 6-7 shows typical photographs of cleaned surfaces of corroded strands and their remaining C_T generated from this test program. The second column in Figure 6-7 shows the definitions of the types of corrosion observed. Figure 6-7(a) shows the surface of an “as-received” strand, which showed a C_T equal to 60.1 kips (267.4 kN). Figure 6-7(b) shows that minor pitting corrosion can cause the C_T to fall below $MUTS_{ARS}$ (i.e., 58.6 to 57.7 kips [260.5 to 256.5 kN]). The observed C_T decreases as more corrosion is observed (see Figure 6-7[c] through [e]). Figure 6-7(f) shows that more localized and pitting corrosion resulted in a greater loss of C_T .



No significant reduction in capacity
Even surface
Low uniform surface corrosion
No pitting corrosion

(a)



2.6% reduction in capacity
Slightly uneven surface
Low localized or pitting corrosion

(b)



7.4% reduction in capacity
Moderately uneven surface
Moderate localized or pitting corrosion

(c)



11.1% reduction in capacity
Highly uneven surface
High localized or pitting corrosion

(d)



12.9% reduction in capacity
Highly uneven surface
High localized or pitting corrosion

(e)



15.3% reduction in capacity
Highly uneven surface
High localized or pitting corrosion

(f)

Notes:

1 kip = 4.45 kN

The percent reduction in capacity is calculated based on an initial capacity of 59.27 kips.

Figure 6-7. Cleaned Strand Surface, Corrosion Characteristics, and Residual Tension Capacity.

6.4.3. Tension capacities of strand and wire specimens

The purpose of this subsection is to present all C_T data generated from the strand and wire corrosion test programs outlined in Subsection 5.5. The data are graphically presented in Figure 6-8 through Figure 6-12. In addition, the same data in tabular format are presented in APPENDIX A. This subsection does not provide any statistical or quantitative analyses of the data. Such analyses are provided through the hypothesis testing in Subsection 6.5 and probabilistic modeling in Section 7.

6.4.3.1. *Unstressed strand capacities under wet-dry exposure conditions*

The experimental design for the unstressed strand corrosion tests under WD conditions was provided in Figure 5-8(a) and Table 5-10 in Subsection 5.5.2.1. Figure 6-8 and Figure 6-9 show the C_T of the unstressed strand specimens after exposure to various corrosive environments. There are thirteen columns in Figure 6-8 and Figure 6-9. The first columns in both Figure 6-8 and Figure 6-9 are identical to the first column in Figure 5-2 and show the C_T of “as-received” strands. The remaining twelve columns are separated into four groups (i.e., 0.006, 0.018, 0.18, and 1.8 %s Cl^- levels) using vertical dashed lines. For each %s Cl^- level, there are three data columns (with hollow, grey-shaded, and black-shaded data markers) corresponding to t_{WD} equal to 12, 16, and 21 months, respectively. The vertical text “Not tested” indicates a parameter combination that was not tested in the program. For example, the unstressed strand specimens were not tested at 16 months and are indicated using the vertical text “Not tested.” The solid horizontal lines in each column indicate the mean values of the C_T .

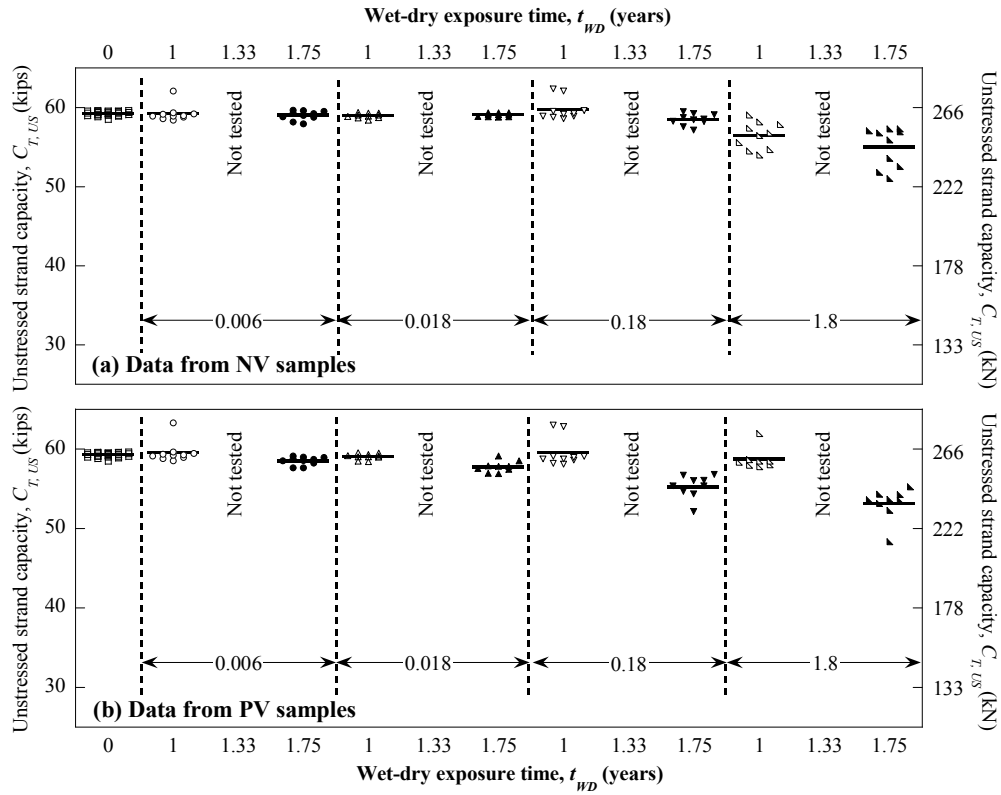


Figure 6-8. Tension Capacities of Unstressed Strands Exposed to NV and PV Conditions.

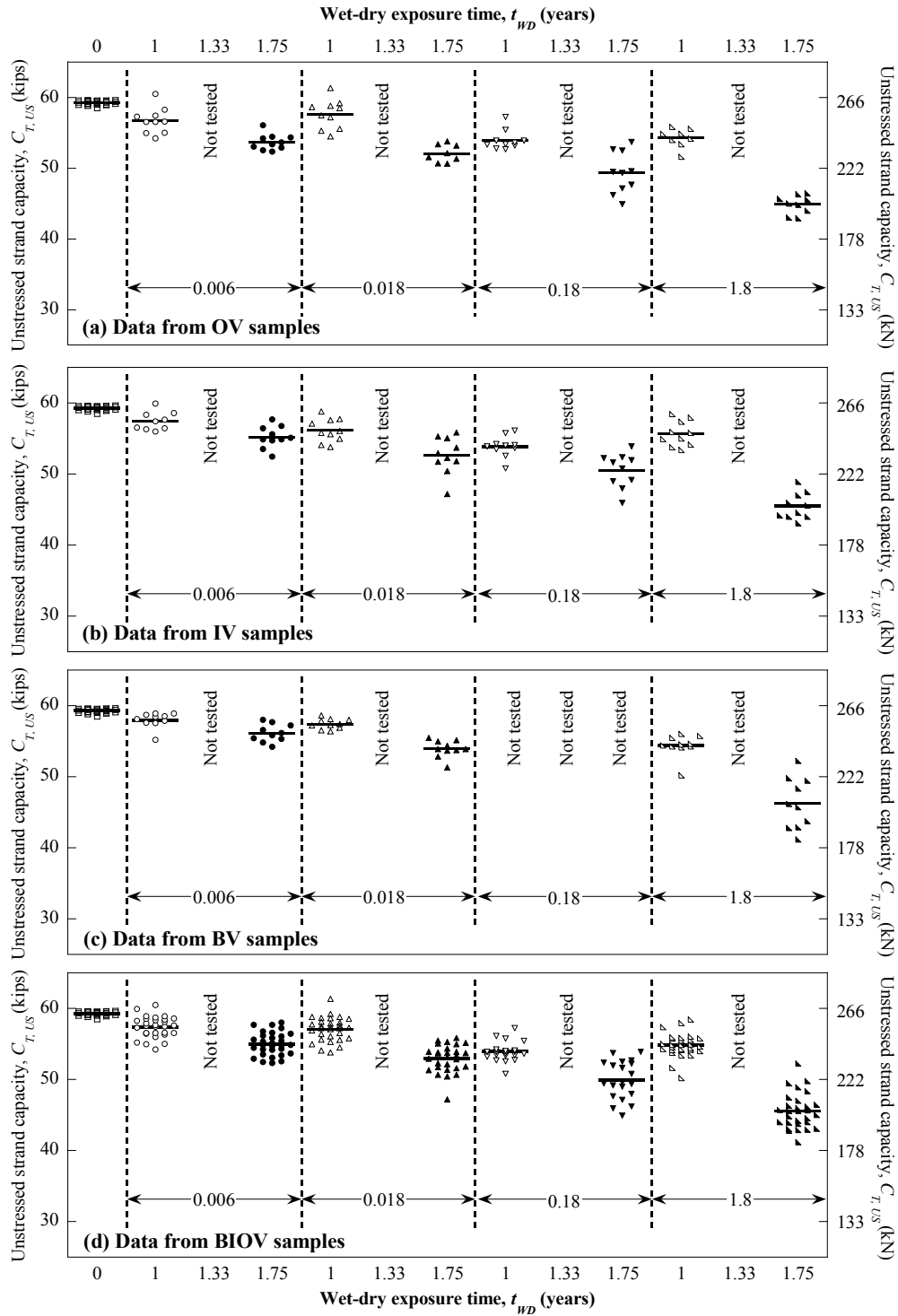


Figure 6-9. Tension Capacities of Unstressed Strands Exposed to OV, IV, BV, and BIOV Conditions.

Figure 6-8(a) and (b) show the C_T data corresponding to NV and PV conditions, respectively. Figure 6-9(a) through (d) show the C_T data corresponding to OV, IV, BV, and BIOV conditions. The BIOV condition and the corresponding data were defined in Subsection 5.5.2.1 and represent all the data obtained from the samples exposed to OV, IV, and BV conditions. This combination will be used to develop the probabilistic models for the BIOV conditions (see Section 7).

6.4.3.2. *Stressed strand capacities under wet-dry exposure conditions*

Figure 6-10 shows the C_T obtained from the stressed strand corrosion tests under WD conditions. The experimental design for this test program was provided in Figure 5-8(b) and Table 5-11 in Subsection 5.5.2.1.

Figure 6-10(a), (b), and (c) show the C_T data obtained from the stressed strands exposed to NV, PV, and OV conditions. The stressed strand specimens were not tested at 0.18 % sCI^- level. Also, the stressed strand specimens were not tested under the IV and BV conditions and, hence, only the data from the OV condition only is used to develop the probabilistic models for BIOV conditions (see Section 7).

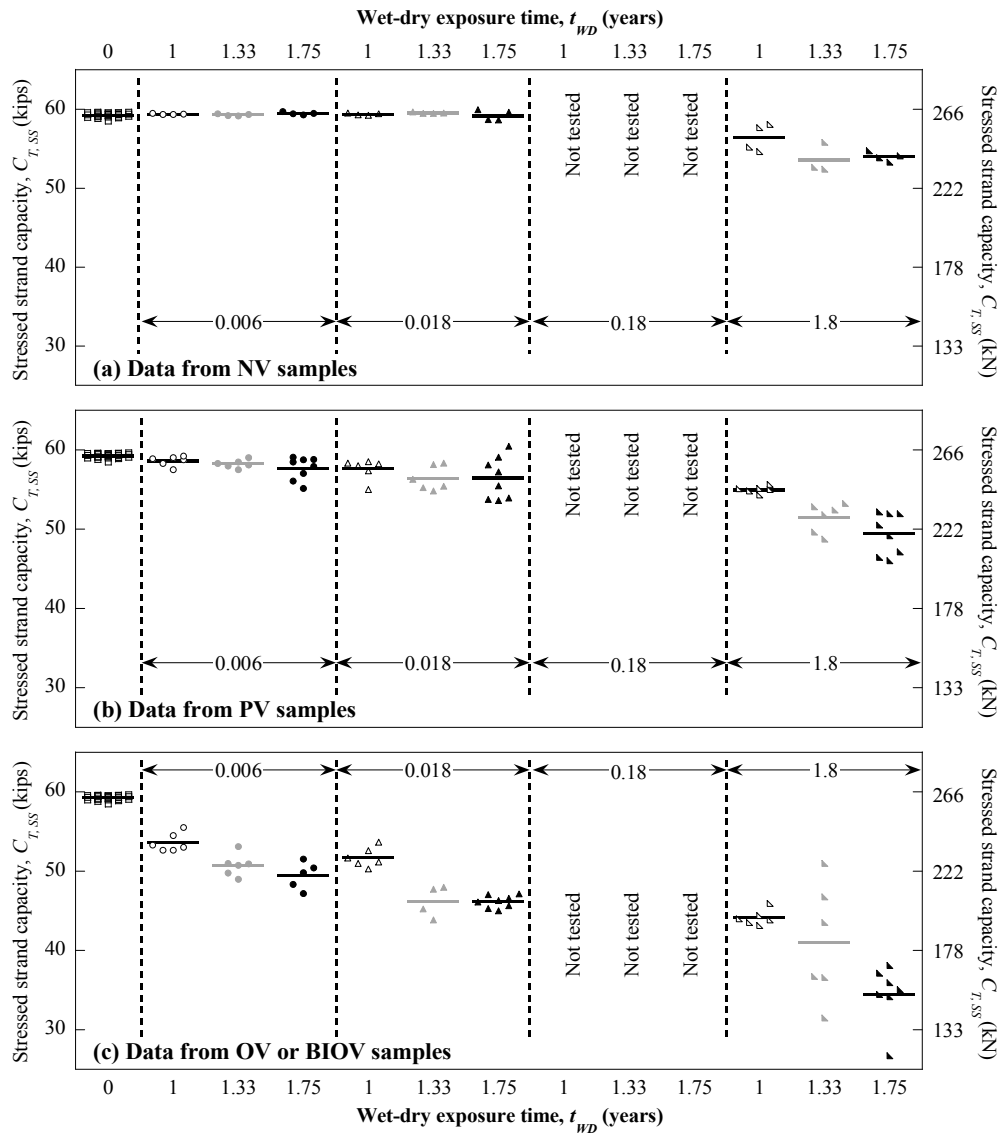


Figure 6-10. Tension Capacities of Stressed Strands Exposed to NV, PV, and BIOV Conditions.

6.4.3.3. Unstressed wire capacities under wet-dry exposure conditions

Figure 6-11 shows the C_T data obtained from the unstressed wire corrosion tests exposed to WD and OV conditions. Figure 5-8(c) and Table 5-12 in Subsection 5.5.2.1 showed the experimental design for this test program. Figure 6-11(a) through (e) show the dot plots with the C_T data obtained from wire specimens exposed to 0.0001, 0.006, 0.018, 0.18, and 1.8 % sCl^- , respectively, for 0, 3, 6, 9, and 12 months of exposure. In Figure 6-11(a) through (e), the first

column is identical to the second column in Figure 5-2 and shows the C_T of the “as-received” wires. This data from the wire samples with OV conditions will be used to develop the probabilistic models for wires under BIOV conditions (see Section 7).

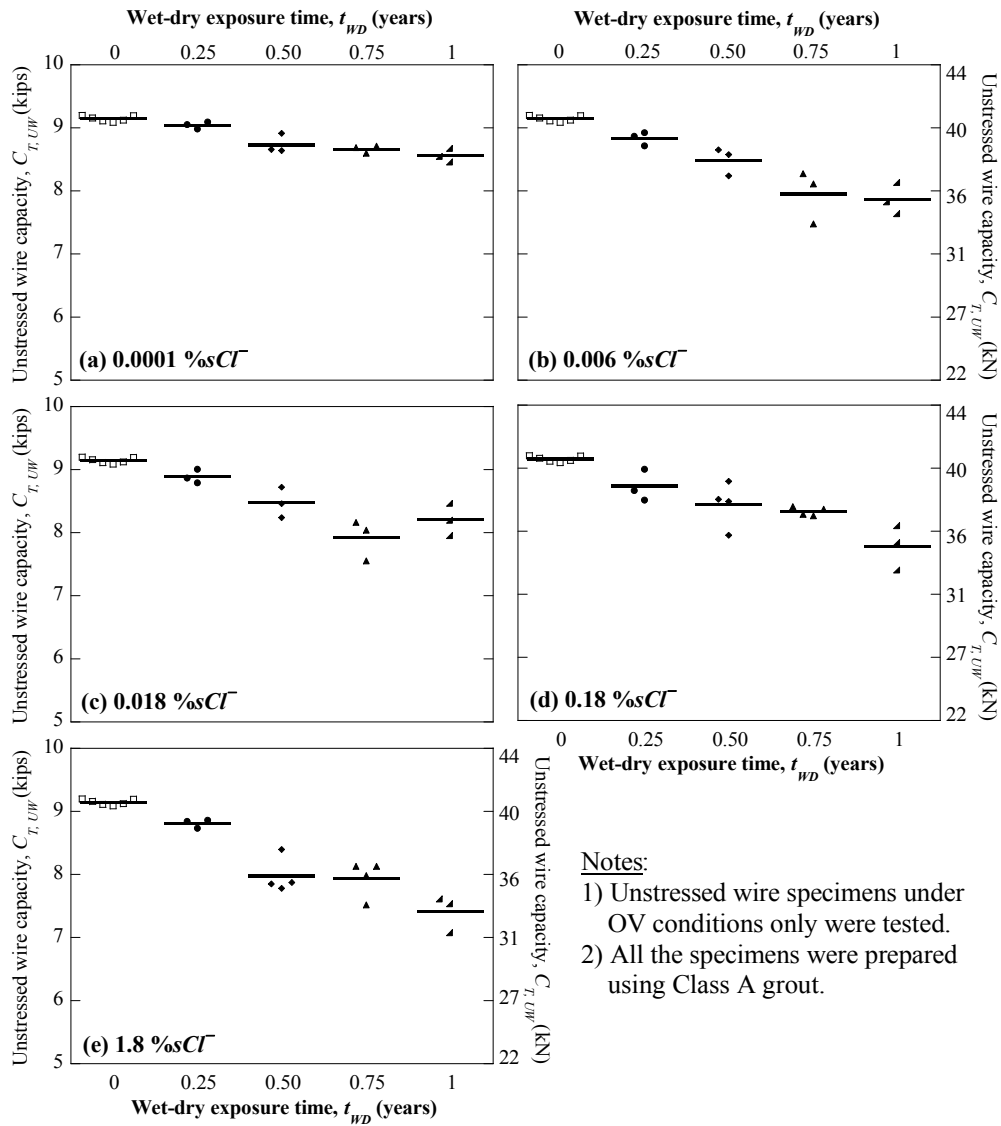


Figure 6-11. Tension Capacities of Unstressed Wires Exposed to WD and OV Conditions.

6.4.3.4. *Unstressed wire capacities under continuous-atmospheric exposure conditions*

Figure 6-12 shows the C_T data obtained from the unstressed wire specimens exposed to the CA and OV conditions. Figure 5-8(c) and Table 5-13 in Subsection 5.5.2.1 showed the experimental design for this test program. The exposure conditions included three levels of T , three levels of RH , and five levels of $\%gCI^-$. However, the experimental design was not full-factorial, and the data corresponding to some parameter combinations are not available. Figure 6-12(a) through (e) show the C_T data obtained from the wire specimens exposed to 0.00001, 0.014, 0.092, 0.343, and 0.782 $\%gCI^-$, respectively, for 9 months of exposure. The abscissa indicates the level of RH . The data corresponding to the three observed levels of T (i.e., 43, 85, and 94°F [6.1, 29.4, and 34.4°C]) is shown using different data markers. For clarity in reading, these data are also aligned vertically with the three forks of the temperature-axis positioned at each observed level of RH (i.e., 45, 70, and 97 percent). Most data points, except the four data points at 94°F (34.4°C), 97 percent RH , and 0.782 $\%gCI^-$ in Figure 6-12(e), showed negligible loss in tension capacity. These data will be used to develop the probabilistic models for unstressed wires under CA and BIOV exposure conditions (see Section 7).

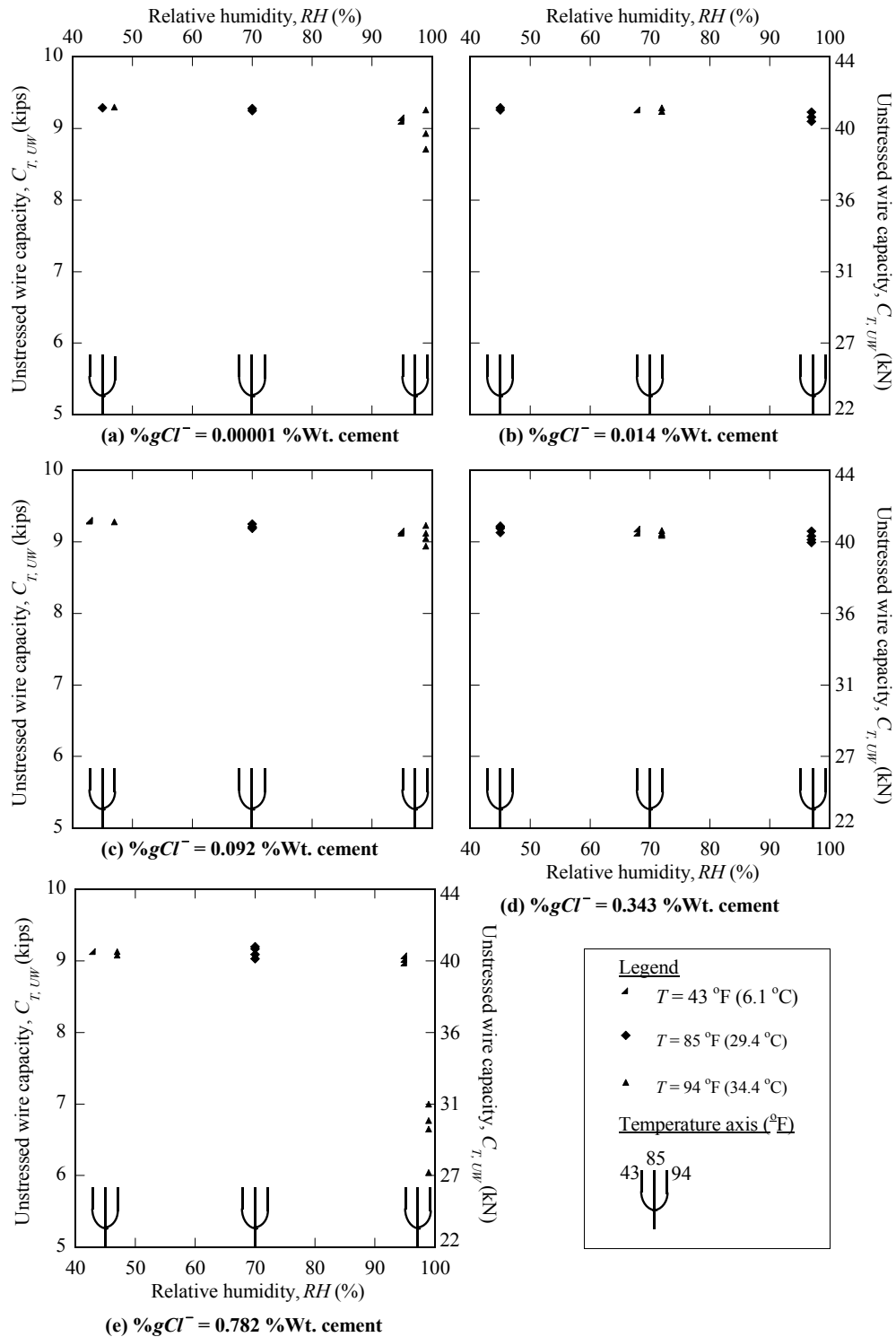


Figure 6-12. Tension Capacities of Unstressed Wires Exposed to CA and OV Conditions.

6.5. CRITICAL PARAMETERS INFLUENCING TENSION CAPACITY OF PT STRANDS

The unstressed and stressed strand corrosion tests were conducted at various factors/levels of test parameters, such as GC , MC , $\%sCl^-$, VT , and f_{axial} (see Figure 5-8[a] and [b] and Table 5-10 and Table 5-11 for experimental designs). At the end of each exposure period, the remaining C_T of unstressed and stressed strands was determined. In this subsection, the experimental data obtained at t_{WD} equal to 12 months are used to identify the critical parameters (or statistically significant parameters) influencing C_T of strands. This is done by performing statistical hypothesis tests on two sample data sets. The two sample data sets for each hypothesis test will have one parameter at different levels/factors and all the other parameters at identical levels/factors. The procedure to select an appropriate hypothesis test is discussed next.

6.5.1. Statistical hypothesis tests

Two independent-sample hypothesis tests are used to check whether a change in a test parameter causes a statistically significant change in C_T of strands. The first step is to determine whether the data sets follow normal distribution using the Kolmogorov-Smirnov normality test. If the sample data sets follow normal distribution, the Student's t -test can be used to check whether a test parameter is significant. If the sample data set does not follow normal distribution, then the Mann-Whitney U (M-W U) test (also called Mann-Whitney-Wilcoxon or Wilcoxon rank sum test) can be used to determine if the test parameter is significant or the two data sets are statistically different. The M-W U hypothesis test is a non-parametric test equivalent to the two-sample Student's t -test. It does not make any assumptions about the data distribution and relies on the ranking of the observations. It can also deal with two independent data sets with small and unequal sample sizes, as is the case of tension capacity data sets from most parameter combinations in the current study. Five percent significance level is used for all the hypothesis tests in this study.

6.5.2. Effect of grout class

The corrosivity of the cementitious grout depends on the qualitative variable, GC , representing the combined effect of the chemical characteristics of the grout material and w/p. The M-W U

hypothesis tests on the C_T data sets with Class A and C grouts (all other variables being identical) resulted in large p-values (≈ 1), indicating no statistically significant difference in tension capacity with the change in GC . This indicates that, at a void location or GAS interface, both Class A and C grouts are associated with similar amounts of corrosion and C_T loss (indicating similar probability of failure). However, in the field, it is likely, based on other studies not reported here (ASBI 2000, FDOT 2001a and b), that Class A (non-thixotropic) grout results in more bleed water and, therefore, a greater number of voids and cracks than Class C (thixotropic and low-bleed) grouts. Hence, given that other variables and conditions remain the same, the tendons with Class A grout will likely have more void or crack locations with more corrosion than tendons with Class C grouts. This may result in a higher joint probability of failure of tendons with Class A grout than those with Class C grout. However, the current research focuses on conditions where a void or GAS interface exists. Because it is determined that GC is not a significant parameter for such conditions, it will not be discussed further.

6.5.3. Effect of moisture and chloride conditions

Figure 6-13 (a), (b), and (c) show the C_T of unstressed and stressed strand specimens. Because IV and BV samples were not exposed to high f_{axial} conditions, Figure 6-13 (d) and (e) show the C_T of only unstressed strand specimens. The data marker in each figure represents a specific combination of VT , MC , and $\%sCl^-$. The round markers indicate low MC level, and diamond and triangular markers indicate high MC levels. Among these, the hollow and solid markers represent the unstressed and stressed conditions, respectively. Also, the letters U and S in the legend indicate unstressed and stressed conditions, respectively. The statistical parameters of 12-month C_T results are also summarized in Table 6-2.

Figure 6-13 and hypothesis tests indicate that the samples exposed to 0.0001 $\%sCl^-$ (i.e., standard room environment with low MC and no WD cycles) did not experience statistically significant reduction in tension capacity over the 12-month exposure period. Samples with WD exposures (i.e., high MC) experienced more corrosion. The samples exposed to 0.006 $\%sCl^-$ (standard water from laboratory sources) experienced severe localized corrosion near the GAS interface but did not experience pitting corrosion because of very small chloride availability. Samples exposed to 0.018, 0.18, and 1.8 $\%sCl^-$ solutions experienced pitting corrosion due to the

presence of higher amounts of chlorides. Based on the current experimental data, the critical chloride threshold for the strands (under direct exposure) is likely between 0.006 and 0.018 % sCI . However, because it seems unnecessary, chloride threshold is not defined and used in this study.

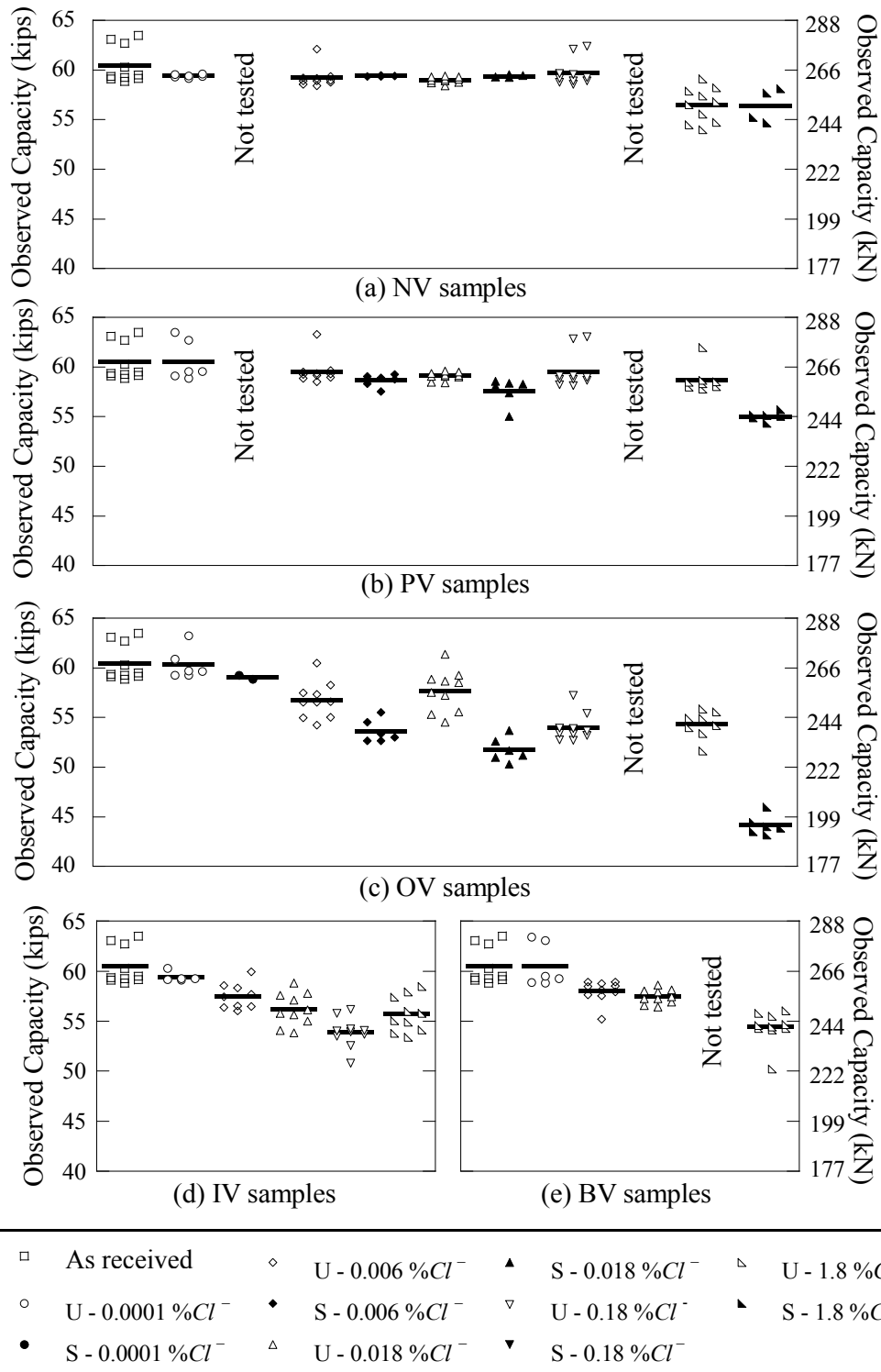


Figure 6-13. Tension Capacity of Unstressed and Stressed Strand Specimens at the End of 12-Months of WD Exposure.

Table 6-2. Mean[‡], Coefficient of Variation (COV), and Mean Capacity Loss ($C_{T,loss}$)* of Tension Capacities Observed at the End of 12-Month WD Exposure.

Axial stress level (f_{axial})	Chloride concentration in exposure solution (% sCl^-)	Statistical parameters	Void type (VT)				
			No void (NV)	Parallel void (PV)	Orthogonal void (OV)	Inclined void (IV)	Bleed water void (BV)
0 ksi (\sim 0 MPa)	0.0001**	Mean (kips)	59.4	60.5	60.33	59.4	60.5
		COV	0.003	0.033	0.026	0.008	0.036
		$C_{T,loss}$ (%)	1.8	0.0	0.2	1.8	0.0
	0.006	Mean (kips)	59.2	59.6	56.8	57.5	58
		COV	0.018	0.023	0.032	0.022	0.019
		$C_{T,loss}$ (%)	2.0	1.5	6.2	5.0	4.1
	0.018	Mean (kips)	59.0	59.1	57.7	56.2	57.5
		COV	0.005	0.006	0.036	0.029	0.013
		$C_{T,loss}$ (%)	2.5	2.3	4.6	7.1	5.0
	0.18	Mean (kips)	59.7	59.6	54.0	53.9	-
		COV	0.023	0.030	0.026	0.028	-
		$C_{T,loss}$ (%)	1.2	1.5	10.8	10.9	-
	1.8	Mean (kips)	56.5	58.7	54.3	55.7	54.4
		COV	0.030	0.023	0.025	0.032	0.030
		$C_{T,loss}$ (%)	6.6	2.9	10.2	7.9	10.1
150 ksi (\sim 1034 MPa)	0.0001**	Mean (kips)	-	-	59.1	-	-
		COV	-	-	0.500	-	-
		$C_{T,loss}$ (%)	-	-	2.3	-	-
	0.006	Mean (kips)	59.4	58.6	53.6	-	-
		COV	0.001	0.011	0.022	-	-
		$C_{T,loss}$ (%)	1.8	3.0	11.4	-	-
	0.018	Mean (kips)	59.4	57.6	51.7	-	-
		COV	0.002	0.023	0.024	-	-
		$C_{T,loss}$ (%)	1.8	4.8	14.4	-	-
	1.8	Mean (kips)	56.4	55.0	44.2	-	-
		COV	0.031	0.008	0.022	-	-
		$C_{T,loss}$ (%)	6.7	9.0	27.0	-	-
Strands with negligible corrosion		Mean (kips)	59.27				
		COV	0.0049				

[‡] 1 kip = 4.45 kN

* $C_{T,loss}$ greater than 0.0049% indicates that the mean C_T is less than the $MUTS_{ARS}$.

** Specimens were kept in standard room conditions and not exposed to wet-dry cycle.

- Indicates that no samples were tested.

For the NV samples, the M-W U hypothesis test exhibited no statistically significant difference in the C_T as the % sCl^- level increased from 0.0001 to 0.018 (with levels/factors of all

other variables being identical). The p-value of the hypothesis test was 0.0085 indicating a statistically significant difference in C_T as the %s Cl^- increased from 0.018 to 1.8 (with levels/factors of all other variables being identical). In the case of the NV samples with 1.8 %s Cl^- , a large amount of chlorides precipitated inside the cracks formed in the 0.25-inch (6 mm) grout cover, and these chlorides were easily transported to the embedded strand surface, causing pitting corrosion. The average C_T values (horizontal bar) in Figure 6-13(d) shows an increase in remaining C_T as the %s Cl^- increased from 0.18 to 1.8. However, this is misleading because the p-value of the M-W U hypothesis test (at 5 percent significance level) between these two data sets was 0.0433, which is very close to the critical value of 0.05. Because $0.0433 \approx 0.05$, it can be concluded, on a rational basis, that the remaining C_T of the IV samples exposed to 0.18 and 1.8 %s Cl^- are statistically similar. Moreover, because of the variability in the corrosion-induced loss in the C_T , conclusions should not be made based only on average values, which do not represent the variability in the behavior even in controlled laboratory experiments. Figure 6-13 shows a general trend of increased corrosion activity for all void types with increased MC and %s Cl^- levels. Also, the M-W U tests indicated that the MC and %s Cl^- are significant parameters influencing C_T .

6.5.4. Effect of stress conditions

The NV, PV, and OV sample conditions were assessed for the effect of stress level. Both unstressed and stressed samples experienced similar reductions in C_T when completely embedded in grout (i.e., the NV samples). This is likely due to the shorter exposure durations used in this research. Also, the p-values obtained in the corresponding M-W U tests were larger than 0.05, indicating no evidence to reject the null hypothesis that the C_T of the NV samples with unstressed and stressed conditions are equal. This behavior is likely due to the passive protection by the uncontaminated and alkaline cementitious grout at the strand surface. When voids were present (i.e., PV and OV samples), the stressed samples exhibited significantly higher losses in the C_T than those exhibited by the corresponding unstressed samples. For example, the M-W U test on the unstressed and stressed samples with PV and OV conditions at 1.8 % Cl^- resulted in a p-value of 0.002. The p-value from the M-W U test on the unstressed and stressed PV samples at 0.006 % Cl^- was 0.034 and that for the M-W U test on unstressed and stressed OV samples at

0.006 % Cl^- was 0.005. A p-value less than 0.05 provides evidence to reject the null hypothesis that the stress level does not cause changes in tension capacity. Hence, the stress level is a statistically significant parameter affecting the corrosion and C_T of strands in void conditions. The general differences in the C_T of the unstressed and stressed specimens were most significant with the OV samples exposed to 1.8 % Cl^- , the condition representing a column or anchorage zones in girders with tendons exposed to chloride environments. The largest mean reduction in C_T occurred on the stressed OV samples exposed to 0.006, 0.018, and 1.8 % Cl^- ; these reductions were 11.4, 14.4, and 27 percent, respectively. The C_T reduction due to high levels of f_{axial} can be attributed to the possible synergistic effects of small surface crack formation at high stress levels, infiltration of moisture and chlorides into these cracks, and resulting accelerated anodic dissolution at these sites.

6.5.5. Effect of void conditions

Of all the strand specimens, the maximum corrosion was observed at or near the GAS interfaces. The OV samples experienced more localized corrosion than the PV samples. For example, the p-values obtained with M-W U tests on the data sets from the OV and PV samples were less than 0.05, except for the room exposure conditions with 0.0001 % sCl^- , where there was no significant change in the C_T . The p-values obtained from M-W U tests for IV and BV data sets with 0.0001, 0.006, 0.018, 0.18, and 1.8 % sCl^- were 1, 1, 0.063, 0.2475, and 1, respectively, indicating no statistically significant change in the C_T as the void condition changed from IV to BV. Moreover, the differences between the mean values of C_T from IV (or BV) samples and OV samples were less than 2.5 percent. As a result, the M-W U tests showed p-values larger than 0.1431, indicating no statistically significant differences in the C_T as the type of void changed from IV (or BV) to OV. This can be explained by the differences in both the geometry and ratio of cathode-anode regions of these test samples. The cathode-to-anode ratio on the OV, IV, BV, and PV samples were 5:1, 5:1, 5:1, and 2:1, resulting in more localized corrosion on the OV, IV, and BV samples than on the PV samples. ASTM G109-07, a longer-term corrosion test method, uses a cathode-to-anode ratio of 2:1. These results indicate that the cathode-anode ratio likely influences the rate of localized corrosion of PT strands at or near the void locations.

Table 6-3 shows the three void groups (i.e., NV, PV, and BIOV), within which there are no statistically significant differences in C_T . Because the maximum corrosion activity and loss in C_T was observed in the BIOV samples, this void group is defined as the governing void group. In summary, VT significantly influences the C_T of strands, and NV, PV, and BIOV conditions have statistically different effects on the loss of C_T .

Table 6-3. Groups of Void Conditions with Similar Effects on Tension Capacity Loss.

Name of void group (as denoted, herein)	Void conditions	Typical location on a PT bridge
NV	No void (NV)	At fully grouted regions on tendons
PV	Parallel void (PV)	At or near midspan of a girder
BIOV	Bleedwater void (BV) Inclined void (IV) Orthogonal void (OV)	At or near anchorage zones of girders At or near high points in columns

6.6. SUMMARY

This section presented the results from the experimental program outlined in Section 1 and the critical parameters influencing tension capacity of strands. In Subsection 6.2, the electrochemical characteristics of ASTM A416 steel were determined using cyclic polarization tests. In Subsection 6.3, galvanic corrosion characteristics of PT systems were determined using the modified ASTM G109 and bearing plate tests. Following this, all C_T data obtained from the strand and wire corrosion tests were presented in Subsection 6.4. Finally, in Subsection 6.5, the critical parameters that affect the C_T of strands were identified using statistical hypothesis tests.

7. MODELING TENSION CAPACITY OF STRANDS EXPOSED TO WET-DRY (WD) CONDITIONS

Equation Section (Next)

7.1. INTRODUCTION

This section first presents statistical procedures to develop probabilistic models. These procedures are used in this section and in Sections 8, 9, and 10. The analytical program to develop the probabilistic models for C_T of strands exposed to WD conditions is then developed. Following this, three sets of models for C_T are formulated and assessed. Then the C_T at future time is predicted using these developed probabilistic models and assumptions based on engineering judgments on potential field exposure conditions.

7.2. STATISTICAL PROCEDURES TO DEVELOP PROBABILISTIC MODELS

This subsection presents the statistical procedures that are used to develop the probabilistic models for C_T of strands.

7.2.1. Statistical diagnosis of experimental data

In this step, possible forms of relationships (i.e., linear, logarithmic, exponential, power, etc.) between the response and predictor parameters are identified. This is done by first developing scatter plots between the basic or transformed response and predictor parameters. These scatter plots are then visually assessed to identify possible relationships and dependencies. This intuitive information is incorporated into the probabilistic model formulation, as discussed next.

7.2.2. Formulation of probabilistic capacity models

Linear or non-linear probabilistic models are formulated considering the identified relationships or patterns from the diagnostic studies and underlying physical reasonings. The C_T of strands

and wires can be modeled using the following several probabilistic model forms developed by Gardoni et al. (2002a and b):

$$R_C(\mathbf{x}, \Theta) = \gamma(\mathbf{x}, \theta) + \sigma\varepsilon \quad (7.1)$$

where $R_C(\mathbf{x}, \Theta)$ is the ratio between the actual C_T of wires or strands, $C_T(\mathbf{x}, \Theta)$, and Minimum Absolute Tensile Strength of “as-received” strands ($MUTS_{ARS}$); $\gamma(\mathbf{x}, \theta)$ is a correction function; \mathbf{x} is a vector of the predictor parameters or regressors; $\Theta = (\theta, \sigma)$ is a vector of unknown model parameters, where $\theta = (\theta_1, \dots, \theta_k)$ is a $1 \times k$ vector of model parameters and k is the number of model parameters considered; and $\sigma\varepsilon$ is the model error, where σ represents the standard deviation of the model error and ε is a random variable with zero mean and unit standard deviation. The term $\gamma(\mathbf{x}, \theta)$ captures two reasons why C_T might be different from $MUTS_{ARS}$: (1) $C_{T,ARS}$ is typically greater than $MUTS_{ARS}$ (see Subsection 5.2.1) because of potential liability concerns with the manufacturer, and (2) corrosion might lead to a reduction in C_T .

To maximize the scope or range of applicability of the probabilistic, models with dimensionless terms (i.e., $R_C(\mathbf{x}, \Theta)$ and $\gamma(\mathbf{x}, \theta)$) are developed. In the models developed in Section 7, the response and most predictor parameters are standardized to form corresponding dimensionless parameters. However, note that the time and temperature parameter have not been standardized due to the lack of an appropriate standardizing factor. Models with dimensionless response or predictor parameters have two benefits over those made of parameters with specific physical dimensions. First, θ associated with dimensionless parameters are also dimensionless. Second, a dimensionless model is applicable when the standardized predictor parameters fall within range of the standardized predictor parameters in the database used to assess the model. This typically expands the range of applicability of a probabilistic model, especially the parameters that are expressed in a dimensionless format.

7.2.3. Assessment or calibration of probabilistic capacity models

7.2.3.1. Posterior statistics of model parameters

The probabilistic models can be written in the following general form:

$$\mathbf{R}_C = \mathbf{G}\boldsymbol{\theta} + \sigma\boldsymbol{\varepsilon} \quad (7.2)$$

where \mathbf{R}_C is an $n \times 1$ vector of standardized tension capacity observations (R_{C1}, \dots, R_{Cn}); n is the number of observations or sample size; \mathbf{G} is an $n \times k$ matrix of known regressors; k is the number of regression parameters (in this study, $k = 4$); and $\boldsymbol{\varepsilon}$ is an $n \times 1$ vector of normal random variables. The remaining quantities are as defined in Equation (7.1).

A closed-form solution for the posterior statistics of $\boldsymbol{\Theta}$ is available in the case of probabilistic models that are linear $\boldsymbol{\Theta}$, for which no prior information is available and no upper or lower bound data are used (Box and Tiao 1992). Box and Tiao (1992) provide expressions for posterior statistics of $\boldsymbol{\Theta}$ as follows:

$$p(\boldsymbol{\Theta} | \mathbf{R}_C) \propto p(\boldsymbol{\Theta}) p(s^2 | \sigma^2) p(\hat{\boldsymbol{\theta}} | \boldsymbol{\theta}, \sigma^2)$$

$$\text{where, } \hat{\boldsymbol{\theta}} = (\mathbf{G}^T \mathbf{G})^{-1} \mathbf{G}^T \mathbf{R}_C \quad (7.3)$$

$$s^2 = \left(\frac{1}{\nu} \right) (\mathbf{R}_C - \hat{\mathbf{R}}_C)^T (\mathbf{R}_C - \hat{\mathbf{R}}_C)$$

$$\nu = n - k, \text{ and}$$

$$\hat{\mathbf{R}}_C = \mathbf{G}\hat{\boldsymbol{\theta}}$$

The marginal posterior distribution of $\boldsymbol{\theta}$ follows a multivariate student t -distribution, $t_k[\hat{\boldsymbol{\theta}}, s^2(\mathbf{G}^T \mathbf{G})^{-1}, \nu]$, where $\hat{\boldsymbol{\theta}}$ is both the mode and the mean of $\boldsymbol{\theta}$, and the covariance matrix is $s^2(\mathbf{G}^T \mathbf{G})^{-1} / (\nu - 2)$. For a given \mathbf{x} , $\boldsymbol{\theta}$, and σ , the model variance, $\text{Var}[R_C(\mathbf{x}, \boldsymbol{\Theta})]$, is equal to σ^2 . Therefore, σ^2 (or σ) can be used as a measure of the model accuracy. The marginal posterior

distribution of σ^2 is $\nu s^2 \chi_\nu^{-2}$, where χ_ν^{-2} is the inverted chi-square distribution with ν degrees of freedom, mean $\nu s^2 / (\nu - 2)$ and variance $2\nu^2 s^4 / [(\nu - 2)^2 (\nu - 4)]$. As an approximation, σ can be calculated as $\sqrt{\sigma^2}$ and used to measure model accuracy.

In the case of non-linear probabilistic models, the test parameters are non-linear, no prior information is available about the distribution of Θ , and no upper or lower bound data are used. In this case, the parameters Θ can be estimated using the updating rule as follows (Box and Tiao 1992):

$$f(\Theta) = \kappa L(\Theta) p(\Theta) \quad (7.4)$$

where $f(\Theta)$ = posterior distribution representing updated state of our knowledge about Θ ; $L(\Theta)$ = likelihood function representing the objective information on Θ contained in a set of observations; $p(\Theta)$ = prior distribution representing state of knowledge about Θ prior to obtaining the observations; and κ is the normalizing factor, which is defined as $\kappa = [\int L(\Theta) p(\Theta) d\Theta]^{-1}$. The mathematical expression for the prior statistics of Θ is as follows (Gardoni et al. 2002a):

$$p(\Theta) \cong p(\Sigma) \propto |\rho|^{-(q+1)/2} \prod_{i=1}^q \frac{1}{\sigma_i} \quad (7.5)$$

where ρ_{ij} is an element of the correlation matrix. If no upper or lower bound data are used and the observations are statistically independent, the likelihood function for the univariate model form is as follows:

$$L(\theta, \sigma) \propto \prod_{data} \left(p(\sigma) \varphi \left[\frac{r_i(\theta)}{\sigma} \right] \right); \text{ where, } p(\sigma) \propto \frac{1}{\sigma}; r_i = i^{th} \text{ residual} \quad (7.6)$$

7.2.3.2. Mean absolute percentage error

In addition to σ^2 and σ , the Mean Absolute Percentage Error (MAPE) can also be used to provide an intuitive measure of model accuracy. The MAPE can be mathematically expressed as follows:

$$\text{MAPE} = \frac{1}{n} \left[\sum_{i=1}^n \left(\frac{|E[C_T(\mathbf{x}_i, \boldsymbol{\theta})] - C_{T,i}|}{C_{T,i}} \right) \right] \times 100 \quad (7.7)$$

where $C_{T,i}$ is the observed tension capacity and $E[C_T(\mathbf{x}_i, \boldsymbol{\theta})] = C_{Tn,ARS} \times \gamma(\mathbf{x}_i, \hat{\boldsymbol{\theta}})$ is the mean predicted capacity.

7.2.3.3. Selection of the most parsimonious parameter set

Although the diagnostic plots can assist in identifying the unimportant and important predictor parameters, their selection needs to be justified for being the most parsimonious parameter set (i.e., as few parameters, θ_i , as possible). In other words, from a statistical standpoint, in order to avoid (1) the loss of precision on the estimates of the parameters and on the overall model (due to the inclusion of statistically redundant variables) and (2) overfitting the data, a model should have parsimonious parameterization. To attain such a model, a stepwise variable deletion process can be carried out on the full model under consideration. Following the general procedure developed by Gardoni et al. (2002a and b), the coefficient of variation (COV) of the parameter estimates and σ for the full and reduced models are compared to determine the most parsimonious model. Engineering judgment could, and should, play a role in final model selection.

7.2.3.4. Possible high correlation coefficients between model parameters

There exist practical difficulties in interpreting the numerical values of the correlation between the model parameters in the probabilistic models developed, especially in the cases of high

positive and high negative correlations. According to Ryan (2007), “When predictor variables are random variables, the multiple regression coefficients should not be simply interpreted as a relationship between the response and corresponding predictor variables....”

7.2.3.5. *Validation plot*

A validation plot is defined as the scatter plot between the observed and predicted values of the response parameter. A validation plot can be used to assess the overall model quality and investigate whether the model agrees, at least approximately, with the homoskedasticity and normality assumptions. Definitions and use of some key elements of the validation plot are defined here.

- “1:1 line” is defined as the solid line (with unit slope) indicating the median predicted capacity values. There is a 50 percent probability for the true capacity to be above or below this line. If the data points fall along the 1:1 line, then the model can be assumed to be of good prediction quality. The pattern and width of the spread of data can be used to assess the bias or residual trend. If data points are spread with an approximately equal width and along the 1:1 line, then the homoskedasticity assumption for the model holds reasonably well. The homoskedasticity assumption means that the variance of the model error is approximately constant and independent of the predictor variables. It should be noted that the more the statistical sample size, the less the difficulty in visually observing the variance of the model error, if any, in the validation plot and thereby less difficulty in assessing the homoskedasticity characteristics.
- “ $\pm\sigma$ region” is defined as the region within one standard deviation from the median predicted capacity. This is indicated by the region within the two dashed lines (with unit slope) that are parallel to and located at one standard deviation away from the 1:1 line. Ideally, if 68.23 percent of data points lie inside this region, then the normality assumption holds good. Although this may not be the case with the capacity models presented in this document, reasonable assumptions on the validity of normality assumption can be made based on the spread of data in and out of the $\pm\sigma$ region. The normality assumption means that the model error follows a normal distribution. It should be noted that the more the statistical sample size, the less the difficulty in visually observing a normally distributed data in the validation plot.
- In most of the validation plots provided in this document, the nominal and yield capacities of pristine strands are indicated using two vertical dashed

lines. Also, for convenience, the abbreviated name of the model being assessed or validated is shown at the top left corner of every validation plot in this document.

- For a perfect prediction model, the predicted and observed capacities should line up along the 1:1 line. However, due to the inherent variability in the corrosion phenomenon, measurement errors, and model errors due to missing variables in Equation(7.1) or an inaccurate model form, there is a scatter around the 1:1 line. In particular, significantly different capacities are observed for test samples that have identical combinations of test parameters. In the validation plots shown in the remaining sections in this document, this variability is evident from the long horizontal stretches of several observed data points corresponding to a particular or same predicted value. Conclusions on “the agreements of the probabilistic models with the homoskedasticity and normality assumptions” may be made based on the consideration that large variability is an inherent property of the electrochemical corrosion process and the resulting capacity loss. In other words, a probabilistic model for capacity of strands or wires may be considered to have good agreement with the homoskedasticity and normality assumptions even though there may be some scatter among the data points along the 1:1 line. This is because the scatter in the data points may be due to the inherent scatter in the corrosion phenomenon and not due to poor model quality.

Statistical procedures to diagnose the experimental data and formulate the probabilistic capacity models have been presented. Also, procedures to estimate the MAPE, posterior statistics of the linear and non-linear model parameters, and validation plots to assess the quality of the capacity models have been presented. These procedures are used to develop the probabilistic models for C_T of strands.

7.3. PROBABILISTIC MODELS FOR TENSION CAPACITY OF “AS-RECEIVED” STRANDS

The first column of Figure 5-2 in Subsection 5.2.1 shows the C_T of 24 “as-received” strands. Based on this data and assuming a lognormal distribution, the $C_{T,ARS}$ can be statistically expressed as follows:

$$C_{T,ARS} \sim \text{LN}\left(E[C_{T,ARS}], \sqrt{\text{Var}[C_{T,ARS}]}\right) \sim \text{LN}(59.27, 0.29) \text{ kips} \quad (7.8)$$

$$\sim \text{LN}(263.7, 1.3) \text{ kN}$$

Herein, Equation (7.8) will be used to probabilistically predict $C_{T,ARS}$.

7.4. TENSION CAPACITY MODELS FOR STRANDS UNDER WD EXPOSURE

CONDITIONS: ANALYTICAL PROGRAM

This subsection presents the program to develop probabilistic models for C_T of strands exposed to WD conditions. Theoretical models for C_T of strands under WD exposure conditions could not be developed or found in literature. Historically, engineers have developed deterministic empirical models when mechanistic modeling is not possible. In this document, probabilistic empirical models are developed using the data obtained from the experimental program presented in Subsection 5.5 and statistical procedures discussed in Subsection 7.2.

Models for NV, PV, and BIOV conditions are developed using the experimental data from strand corrosion tests with WD exposure conditions. BIOV indicates the group of BV, IV, and OV. These are grouped because all these void types (*VTs*) have statistically similar effects on C_T .

The main objectives of this subsection are to:

1. present the basic and standardized parameters used in the probabilistic models for tension capacity (see Subsection 7.4.1 for details), and
2. present analytical steps to develop probabilistic tension capacity models for strands under WD exposure conditions (see Subsection 7.4.2 for details).

To provide the reader with a sense of expected results, preliminary discussions of possible probabilistic model forms are also provided in this section. The developed probabilistic models are presented in Section 7.5. The parameters used in these probabilistic models are presented next.

7.4.1. Parameters used in the models for wet-dry conditions

Subsection 7.2.2 suggested using dimensionless response parameters and their explanatory or predictor parameters to improve the range of application of probabilistic models (Gardoni et al. 2002b). Based on this suggestion, the response and predictor parameters, except

t_{WD} , t_{CA} , and T , are standardized. The following equations define these basic response and predictor parameters, which are used in probabilistic modeling of C_T .

$$R_{C_{T,US}} = \frac{\text{Tension capacity of unstressed strand}}{MUTS_{ARS}} = \frac{C_{T,US}}{MUTS_{ARS}} \quad (7.9)$$

$$R_{C_{T,SS}} = \frac{\text{Tension capacity of stressed strand}}{MUTS_{ARS}} = \frac{C_{T,SS}}{MUTS_{ARS}} \quad (7.10)$$

$$\begin{aligned} \gamma_{t_{WD}} &= \left(\frac{\text{Average wet-time in a year (months)}}{12 \text{ (months)}} \right) \times \text{Total wet-dry exposure time (years)} \\ &= \phi_{wet} \times t_{WD} \text{ (years); } 0 \leq \phi_{wet} \leq 1 \end{aligned} \quad (7.11)$$

$$\gamma_{\%sCl^-} = \frac{\%Cl^- \text{ in the exposure solution}}{\%sCl^-_{\text{saturated chloride solution}}} = \frac{\%sCl^-}{35.7} \quad (7.12)$$

Using suitable transformations (such as exponential, logarithmic, etc.), if needed, of the basic response and predictor parameters defined and the procedures provided in Subsection 7.2, probabilistic models for C_T of strands under WD and CA exposure conditions are developed. The next three subsections discuss the analytical steps involved in this process.

7.4.2. Analytical steps to develop capacity models for strands under wet-dry conditions (steps WD-1 through WD-5)

Table 7-1 provides a list of nine probabilistic strand capacity models that are developed for WD exposure conditions. The subscripts in each model name indicate the exposure conditions. Model $US_{WD,NV}$, $US_{WD,PV}$, and $US_{WD,BIOV}$ are commonly referred to as Model US_{WD} . Similarly, Model $SS_{WD,NV}$, $SS_{WD,PV}$, and $SS_{WD,BIOV}$ are commonly referred to as Model SS_{WD} . Also,

Model $US-SS_{WD, NV}$, $US-SS_{WD, PV}$, and $US-SS_{WD, BIOV}$ are commonly referred to as Model $US-SS_{WD}$.

Table 7-1. List of Tension Capacity Models for Strands under WD Exposure Conditions.

Expanded model name	Abbreviations for specific models	Abbreviations for a group of models
Unstressed Strand _{Wet-Dry, No Void}	$US_{WD, NV}$	US_{WD}
Unstressed Strand _{Wet-Dry, Parallel Void}	$US_{WD, PV}$	
Unstressed Strand _{Wet-Dry, Bleedwater, Inclined, and Orthogonal Void}	$US_{WD, BIOV}$	
Stressed Strand _{Wet-Dry, No Void}	$SS_{WD, NV}$	SS_{WD}
Stressed Strand _{Wet-Dry, Parallel Void}	$SS_{WD, PV}$	
Stressed Strand _{Wet-Dry, Bleedwater, Inclined, and Orthogonal Void}	$SS_{WD, BIOV}$	
Unstressed Strand-Stressed Strand _{Wet-Dry, No Void}	$US-SS_{WD, NV}$ (or $C_{T, WD, NV}$)	$US-SS_{WD}$ (or $C_{T, WD}$)
Unstressed Strand-Stressed Strand _{Wet-Dry, Parallel Void}	$US-SS_{WD, PV}$ (or $C_{T, WD, PV}$)	
Unstressed Strand-Stressed Strand _{Wet-Dry, Bleedwater, Inclined, and Orthogonal Void}	$US-SS_{WD, BIOV}$ (or $C_{T, WD, BIOV}$)	

The flowchart in Figure 7-1 provides an overview of the steps used to develop the tension capacity models for PT strands under WD exposure conditions. The five boxes with thick border lines in Figure 7-1 indicate the five main steps. Each of these boxes also contains a smaller and inner box with rounded corners, which provides an indication of the result from the corresponding steps. These steps are numbered step #WD-1 through #WD-5. Graphical representations provided in Figure 7-2 will also be used, as needed, in the following discussion.

Step #WD-1

Based on statistical hypothesis tests on the data from unstressed strand corrosion tests, identify groups of void types that cause statistically similar capacity loss.

Void type groups identified:

- 1) No Void (NV)
- 2) Parallel Void (PV)
- 3) Bleedwater Void (BV), Inclined Void (IV), and Orthogonal Void (OV)

Note: The third group is denoted as “BIOV”

Step #WD-2

For each void type group, develop US_{WD} models using the observed unstressed strand capacity and other test parameters.

$$R_{C_{T,US}}(\mathbf{x}, \Theta) = \theta_0 + \theta_1 \gamma_{t_{WD}} + \theta_2 \gamma_{\%sCI} \gamma_{t_{WD}} + \sigma \varepsilon \quad (\text{for NV condition})$$

$$R_{C_{T,US}}(\mathbf{x}, \Theta) = \theta_0 + \theta_1 \gamma_{t_{WD}} + \theta_2 \ln[\gamma_{\%sCI}] \gamma_{t_{WD}} + \sigma \varepsilon \quad (\text{for PV and BIOV conditions})$$

Step #WD-3

For each void type group, develop SS_{WD} models using the observed stressed strand capacity and other test parameters.

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) = \theta_0 + \theta_1 \gamma_{t_{WD}} + \theta_2 \gamma_{\%sCI} \gamma_{t_{WD}} + \sigma \varepsilon \quad (\text{for NV condition})$$

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) = \theta_0 + \theta_1 \gamma_{t_{WD}} + \theta_2 \ln[\gamma_{\%sCI}] \gamma_{t_{WD}} + \sigma \varepsilon \quad (\text{for PV and BIOV conditions})$$

Step #WD-4

For each void type group, develop $US-SS_{WD}$ models using the observed stressed strand capacity and predicted unstressed strand capacity.

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) = \theta_0 \left(\gamma_{C_{T,US}} \right)^{\theta_1} + \sigma \varepsilon \quad (\text{for NV, PV, and BIOV conditions})$$

Step #WD-5

Select an appropriate set of models from US_{WD} , SS_{WD} , and $US-SS_{WD}$ models (by considering model qualities, statistical sample size, flexibility for improvement etc.).

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) = \theta_0 \left(\gamma_{C_{T,US}} \right)^{\theta_1} + \sigma \varepsilon \quad (\text{for NV, PV, and BIOV conditions})$$

Re-define the selected set of models as $C_{T,WD}$ models for reliability studies.

Figure 7-1. Flowchart for Developing the Tension Capacity Models for Strands in WD Conditions.

Step WD-1: Identify three groups of VTs (i.e., NV, PV, and BIOV) with statistically significant differences in their influences on C_T (see first box in Figure 7-1). This is done by conducting statistical hypothesis tests on C_T data obtained from strand samples exposed to different VTs and identical levels of other test parameters. Detailed results from this step are provided in Subsection 7.5.1.

Step WD-2: Develop probabilistic models for $C_{T,US}$ (i.e., US_{WD} models) using the data from unstressed strand corrosion tests (see second box in Figure 7-1 and Figure 7-2[a]). The experimental design was shown in Figure 5-8(a) and Table 5-10. For convenience, this is reproduced in Figure 7-2(a), where the unstressed strand samples inside the rectangular box and “as-received” samples tested at t_{WD} equal to 0 months indicate the data used for Step WD-2. In particular, Models $US_{WD,NV}$ and $US_{WD,PV}$ will be developed using the data from unstressed NV and PV samples, respectively. Model $US_{WD,BIOV}$ will be developed by combining all the data from unstressed BV, IV, and OV samples. In Figure 7-1, the box with rounded corners indicates the possible linear model forms for Model US_{WD} . Results from this step are provided in Subsection 7.5.2.

Step WD-3: Develop probabilistic models for $C_{T,SS}$ (i.e., SS_{WD} models) using the data from stressed strand corrosion tests (see third box in Figure 7-1 and Figure 7-2[b]). The experimental designs were shown in Figure 5-8(b). For convenience, this is reproduced in Figure 7-2(b), where the stressed strand samples inside the inner rectangular box and the data from “as-received” samples tested at t_{WD} equal to 0 months indicate the data used for Step WD-3. In particular, Models $SS_{WD,NV}$, $SS_{WD,PV}$, and $SS_{WD,BIOV}$ will be developed using the data from stressed NV, PV, and OV samples, respectively. (IV and BV conditions have statistically similar influence on C_T of strands. Because of this, the model based on the data from OV samples is applicable to IV and BV samples and, hence, referred to as Model $SS_{WD,BIOV}$.) In Figure 7-1, the inner box with rounded corners indicates two linear forms for Model SS_{WD} . Results from this step are provided in Subsection 7.5.3.

Step WD-4: Develop another set of probabilistic models for $C_{T,SS}$ (i.e., $US-SS_{WD,NV}$ models) using the $C_{T,US}$ predicted using US_{WD} models and the $C_{T,SS}$ observed from stressed strand corrosion tests (see fourth box in Figure 7-1 and Figure 7-2[b] and [c]). These predicted $C_{T,US}$

shown in Figure 7-2(c), observed $C_{T,SS}$ shown in Figure 7-2(b), and $C_{T,ARS}$ are used in Step WD-4. Probabilistic models are developed for NV, PV, and BIOV conditions and are denoted as Model US-SS_{WD,NV}, US-SS_{WD,PV}, and US-SS_{WD,BIOV}, respectively. In Figure 7-1, the inner box with rounded corners indicates a non-linear power model form for US-SS_{WD} models. Results from this step are provided in Subsection 7.5.4.

Step WD-5: Select a suitable set of models from US_{WD}, SS_{WD}, and US-SS_{WD} models developed in steps #WD-2, #WD-3, and #WD-4, respectively (see fifth box in Figure 7-1). This selection will be based on the prediction quality of the models, sample size of the capacity data set used, and ease with which the model can be improved by future researchers. The inner box with rounded corners indicates that the US-SS_{WD} models developed in Step WD-4 are selected and re-defined as $C_{T,WD}$ models. The $C_{T,WD,NV}$ and $C_{T,WD,BIOV}$ models will be used in the modeling and analysis of structural reliability of PT bridges. Results from this step are provided in Subsection 7.5.5.

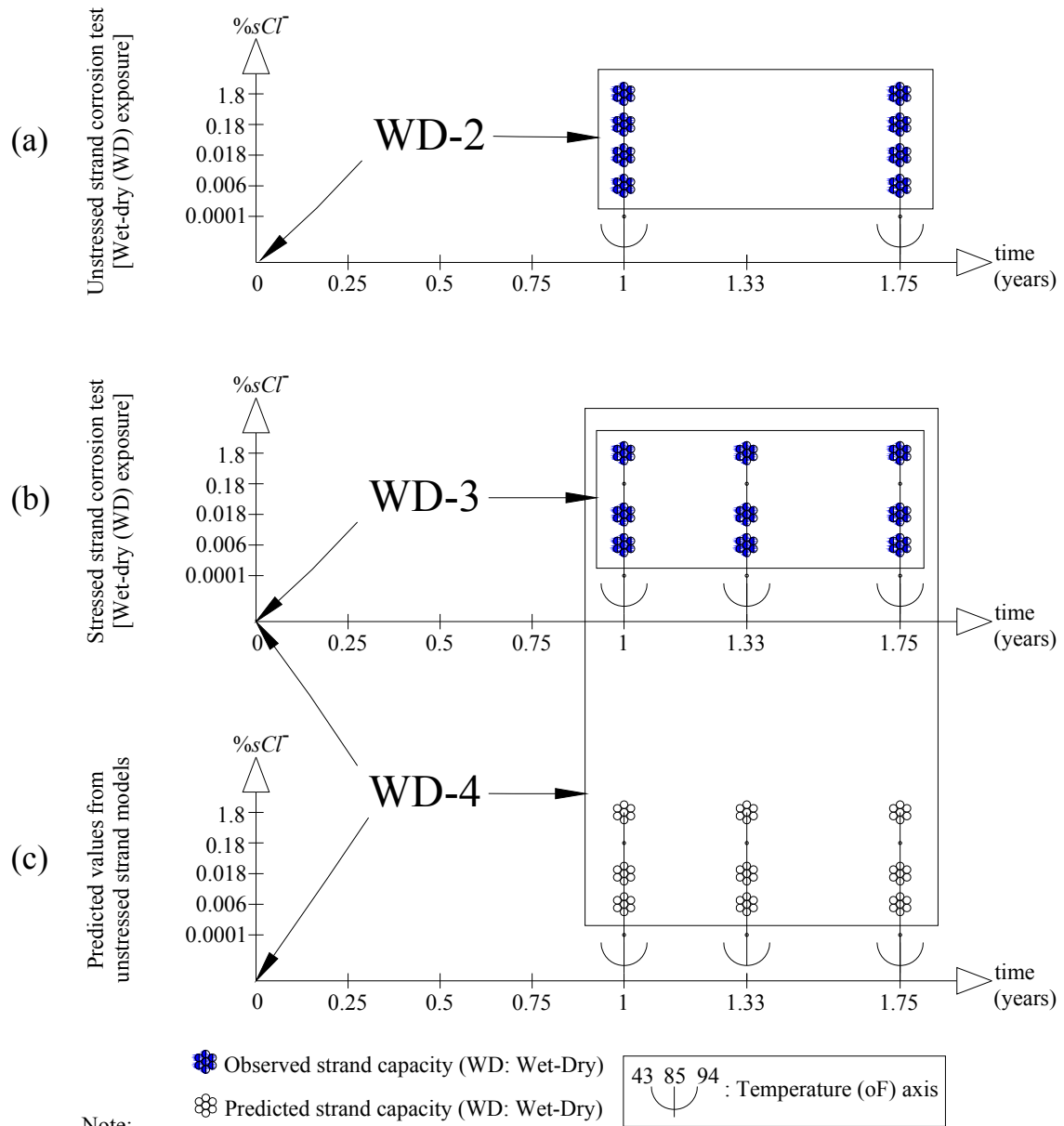


Figure 7-2. Graphical Representations of Analytical Steps WD-2, WD-3, and WD-4.

7.4.3. Assumptions for predicting strand capacity under wet-dry conditions

In Subsection 7.4, US_{WD} , SS_{WD} , and $US-SS_{WD}$ models are developed and, as a demonstration of their applications, C_T of strands is predicted. One of the parameters in the US_{WD} models is ϕ_{wet} (defined in Equation (7.11) in Subsection 7.4.1). As defined in Subsection 5.5.4, the laboratory WD exposure consisted of 14 wet days followed by 14 dry days and, hence, corresponds to ϕ_{wet} of 0.5. However, this may not be the case in the field. Moreover, ϕ_{wet} can vary from one tendon to another on the same bridge and is very complex to estimate. However, based on the literature review in Subsection 2.3.7, practical assumptions, and engineering judgment, an estimation of ϕ_{wet} is provided next.

In this analysis, a rain-day is defined as a day with non-zero precipitation. Note that each data marker in Figure 2-13 indicates a rain-day. The time required for the tendons to automatically dry is defined as t_{drying} . A wet-day (for the tendon) is defined as either a rain-day or a day with at least one rain-day for a period of t_{drying} prior to the day under consideration. Figure 7-3 shows the number of wet-days, calculated based on the above-mentioned definitions and the data shown in Figure 2-13.

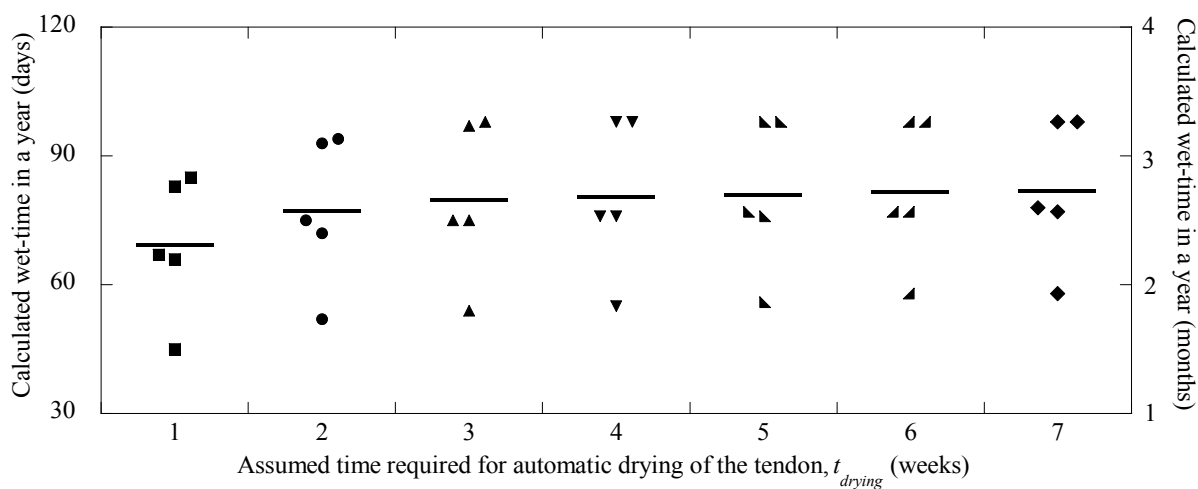


Figure 7-3. Annual Wet-Time for Tendons (Based on Assumed Values of Time Required to Dry the Tendons).

In Figure 7-3, each column corresponds to an assumed value of t_{drying} (i.e., 1, 2, 3, 4, 5, 6, and 7 weeks). The five data markers in each column indicate the number of wet-days in 2001,

2002, 2003, 2004, and 2005. The solid-horizontal line in each column indicates the mean number of wet-days for the corresponding t_{drying} . Based on this data for the downtown region in San Antonio, Texas, the average number of wet-days varies from 69 to 82. The corresponding values of ϕ_{wet} are 0.19 and 0.22.

For all the capacity predictions provided in Subsection 7.4, ϕ_{wet} is assumed to be 0.17, which corresponds to an assumption of 2 months of wet-time in a period of t_{WD} equal to 12 months (i.e., $2/12 = 0.17$). Also, this value falls in the above estimated range (i.e., between 0.19 and 0.22) and, hence, is justified.

7.5. TENSION CAPACITY MODELS FOR STRANDS UNDER WD EXPOSURE

CONDITIONS: RESULTS

The analytical steps (WD-1 through WD-5) to develop probabilistic models for C_T of strands under WD exposure conditions were outlined in Subsection 7.4.2. These analytical steps are executed using the data presented in Subsection 6.4.3 and are presented next.

7.5.1. Step WD-1: Identifying groups of void types with statistically dissimilar effects on tension capacity

NV, PV, and BIOV were identified as three void groups with statistically dissimilar effects on C_T of strands. Therefore, three probabilistic models (i.e., one each for NV, PV, and BIOV conditions) will be developed. Further details on this study were provided earlier in Subsection 6.5.5.

7.5.2. Step WD-2: Models for unstressed strands under WD exposure conditions (using unstressed strand data only)

This subsection presents probabilistic models for $C_{T,US}$ exposed to NV, PV, and BIOV conditions along with WD exposure conditions. These three models are named as $US_{WD,NV}$, $US_{WD,PV}$, and $US_{WD,BIOV}$ models, respectively. Experimental data from 100, 105, and 239 unstressed strand specimens have been used to develop the $US_{WD,NV}$, $US_{WD,PV}$, and $US_{WD,BIOV}$, respectively.

These C_T data were shown earlier in Figure 6-8 and Figure 6-9 in Subsection 6.4.3.1. Prior to discussing model formulation and assessments, statistical diagnosis on the effects of t_{WD} , $\%sCl^-$ and $\ln[\%sCl^-]$ on $C_{T,US}$ are presented.

7.5.2.1. *Diagnosis of the effect of wet-dry exposure time*

Figure 7-4 shows the scatter plot between $C_{T,US}$ and t_{WD} for all the void conditions. The following observations are made based on Figure 7-4(a), (b), and (c) for NV, PV, and BIOV conditions, respectively. A linear relationship seems to exist between $C_{T,US}$ and t_{WD} when NV conditions exist. However, this behavior is more predominant with the samples with higher chloride conditions. In the case of PV conditions, the rate of loss of $C_{T,US}$ seems to be almost 0 until 12 months, after which this rate increases. When BIOV condition exists, a linear relationship between $C_{T,US}$ and t_{WD} seems to be appropriate. Also, a general trend of increasing scatter is observed in Figure 7-4. Because of this, a model with only linear time function might be heteroskedastic. This heteroskedasticity can be minimized by incorporating a two-way interaction term (later discussed in Subsection 7.5.2.3) between t_{WD} and $\%sCl^-$ or $\ln[\%sCl^-]$. The main effects of $\%sCl^-$ and $\ln[\%sCl^-]$ terms on $C_{T,US}$ are discussed next.

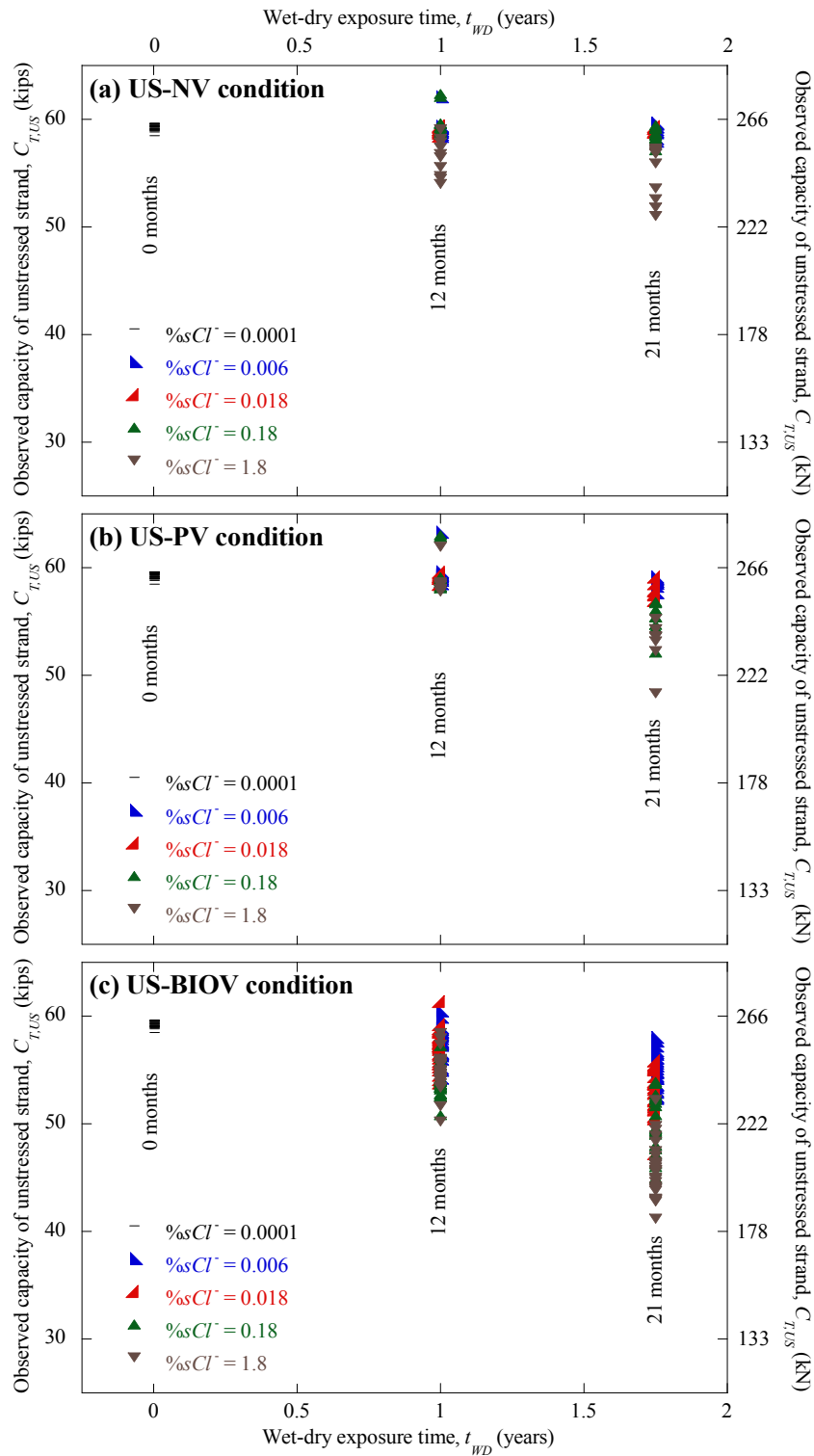


Figure 7-4. Effect of Wet-Dry Exposure Time on Capacity of Unstressed Strands (with $\phi_{wet} = 0.5$).

7.5.2.2. *Diagnosis of the effect of chloride concentration in exposure solution*

The scatter plots in Figure 7-5 and Figure 7-6 show the effect of $\%sCl^-$ and $\ln[\%sCl^-]$ on $C_{T,US}$. In Figure 7-5 and Figure 7-6, the vertical texts next to the data points indicate the actual $\%sCl^-$ values. In Figure 7-5, the data markers for samples corresponding to 0.0001, 0.006, and 0.018 $\%sCl^-$ are closely overlapping along the abscissa. However, this overlapping is avoided using a logarithmic transformation (see Figure 7-6).

A comparison between Figure 7-5(a) and Figure 7-6(a) indicates that, under NV conditions, a more appropriate linear relationship exists between $\%sCl^-$ and $C_{T,US}$ than between $\ln[\%sCl^-]$ and $C_{T,US}$. In addition, in Figure 7-6(a), the slope is almost 0 until 0.18 $\%sCl^-$, after which there is a slight increase in the slope. This is probably because a higher $\%sCl^-$ level than 0.18 was necessary to build sufficient amount of chlorides at the surface of strands (via diffusion through uncracked grout or transport through cracks in the grout cover) and then initiate/propagate corrosion within the test period. In Figure 7-5(b), for PV conditions, the slope significantly reduces after 0.18 $\%sCl^-$, especially with the samples at t_{WD} equal to 21 months. In Figure 7-6(b), the slope seems to be the same throughout the range of abscissa (i.e., $\ln[\%sCl^-]$) considered. This comparison between Figure 7-5(b) and Figure 7-6(b) indicates that $\ln[\%sCl^-]$ term is more appropriate than $\%sCl^-$ term to predict $C_{T,US}$ under PV conditions. Similarly, in Figure 7-5(c), the slope seems to be flattening at or after 0.18 $\%sCl^-$ level. In Figure 7-6(c), the slope seems to be constant throughout the considered range of $\ln[\%sCl^-]$. Hence, the $\ln[\%sCl^-]$ is a more appropriate term than $\%sCl^-$ to predict the $C_{T,US}$ under BIOV conditions. A general trend of increasing scatter as the $\%sCl^-$ increases is observed in Figure 7-5 and Figure 7-6. The model heteroskedasticity that can be induced because of this increasing scatter can be reduced by incorporating an interaction term between t_{WD} and $\%sCl^-$ or $\ln[\%sCl^-]$.

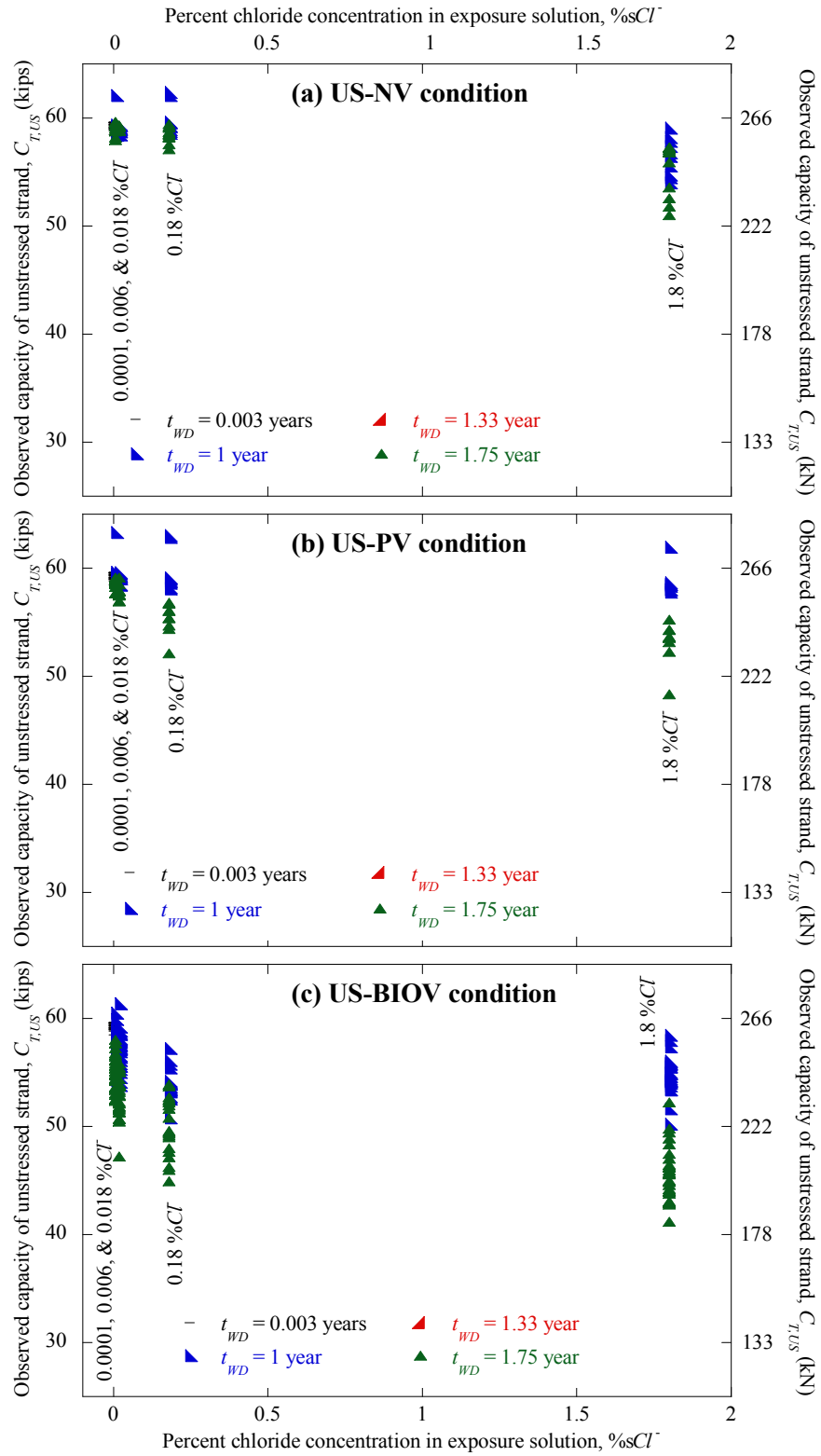


Figure 7-5. Effect of %sCl⁻ on Capacity of Unstressed Strands (with $\phi_{wet} = 0.5$).

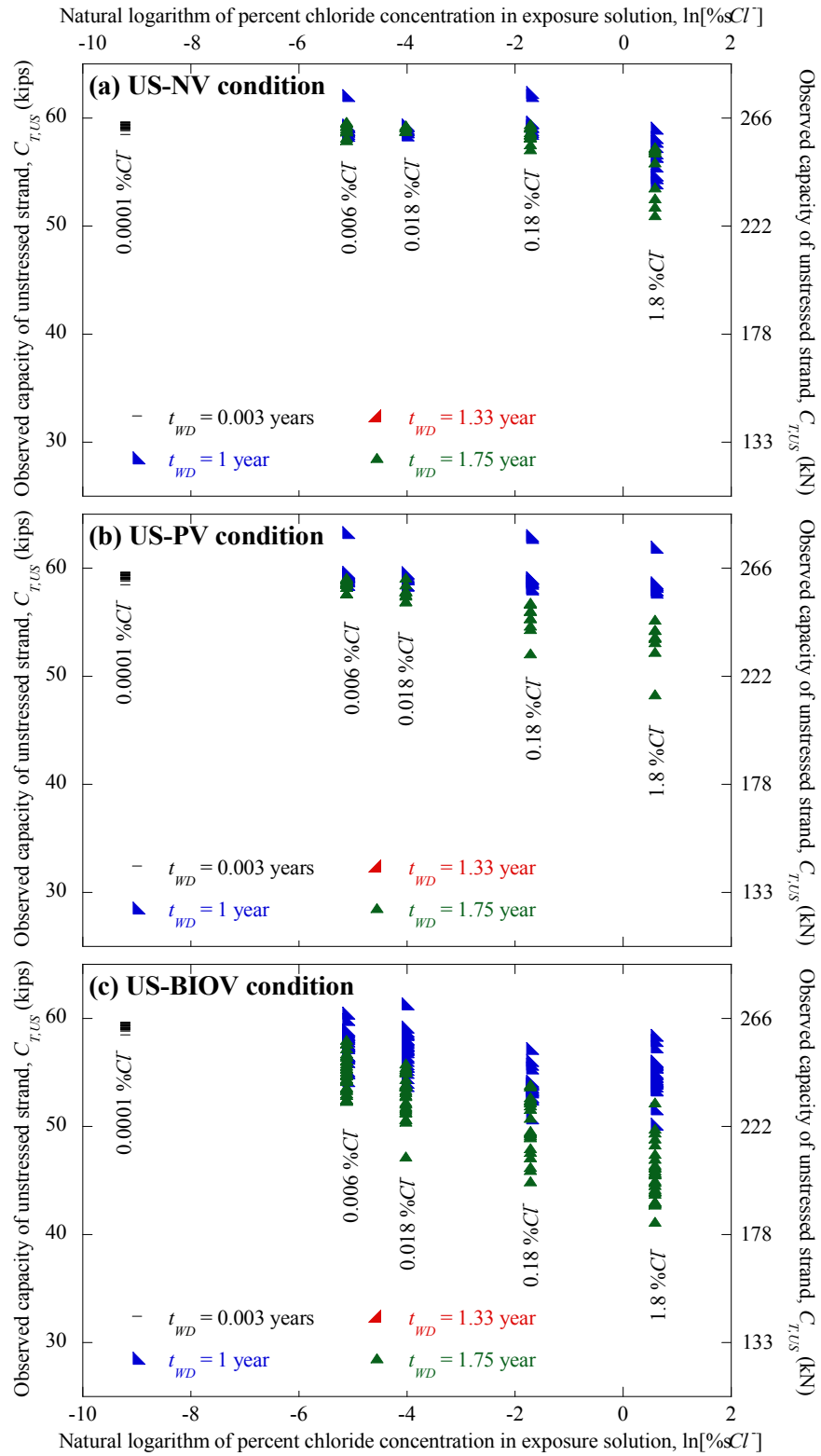


Figure 7-6. Effect of $\ln[\%sCI]$ on Capacity of Unstressed Strands (with $\phi_{wet} = 0.5$).

7.5.2.3. *Diagnosis of the effect of interaction between chloride and time parameters*

In a statistical point of view, main effects of $\%sCl^-$ or $\ln[\%sCl^-]$ terms seem to be good for incorporating into the probabilistic models for $C_{T,US}$. However, based on a corrosion point of view, these main effects are not useful to accurately predict the time-dependent corrosion process. For example, a high t_{WD} value alone with very low $\%sCl^-$ and low moisture condition, or both (for instance, negligible amounts of chlorides or water for 100 years), or a high $\%sCl^-$, high moisture condition, or both, with negligible t_{WD} (for instance, seawater for 1 day) do not normally lead to significant corrosion and reduction in $C_{T,US}$. A two-way interaction term between chloride and time parameters is needed to improve the prediction accuracy and homoskedasticity of the probabilistic models.

In Subsection 7.5.2.2, it was found that the appropriate main effect term for NV conditions is $\%sCl^-$. Hence, the appropriate interaction term for the model for NV conditions is $\%sCl^- \times t_{WD}$. It was also found that the appropriate main effect term for the models for PV and BIOV conditions is $\ln[\%sCl^-]$. Hence, the appropriate interaction term for the models for PV and BIOV conditions is $\ln[\%sCl^-] \times t_{WD}$.

7.5.2.4. *Model formulation*

Based on the diagnostic studies, the probabilistic models for $C_{T,US}$ are formulated and parameters are assessed for each void type. These models are expressed as follows:

$$\begin{aligned} R_{C_{T,US}}(\mathbf{x}, \Theta) &= \theta_0 + \theta_1 \gamma_{t_{WD}} + \theta_2 \gamma_{\%sCl^-} \gamma_{t_{WD}} + \sigma \mathcal{E} \quad (\text{for NV condition}) \\ R_{C_{T,US}}(\mathbf{x}, \Theta) &= \theta_0 + \theta_1 \gamma_{t_{WD}} + \theta_2 \ln[\gamma_{\%sCl^-}] \gamma_{t_{WD}} + \sigma \mathcal{E} \quad (\text{for PV and BIOV conditions}) \end{aligned} \quad (7.13)$$

where the terms are as defined in Equation (7.1) and Subsection 7.4.1. Note that the appropriate interaction term for $US_{WD,NV}$ model is $\gamma_{\%sCl^-} \gamma_{t_{WD}}$ and for $US_{WD,PV}$ or $US_{WD,BIOV}$ models is $\ln[\gamma_{\%sCl^-}] \gamma_{t_{WD}}$. When t_{WD} is 0, the model should predict a capacity that is close to $MUTS_{ARS}$. The intercept term will help meet this criterion. As the corrosion process is a time-dependent

phenomenon, each predictor parameter (except the intercept term) has to be a function of a time parameter (i.e., $\gamma_{t_{WD}}$). The two-way interaction term (i.e., $\gamma_{\%sCl^-} \gamma_{t_{WD}}$ or $\ln[\gamma_{\%sCl^-}] \gamma_{t_{WD}}$) will capture the effect of interaction between the chlorides and time parameters and thereby minimize the heteroskedasticity of the model. Following is a sequential and detailed discussion on the assessment of the three probabilistic models (i.e., Equation (7.13)) for $C_{T,US}$.

7.5.2.5. Model assessment - No Void conditions

MAPE and posterior statistics of the model form for NV conditions (provided in Equation (7.13)) were assessed using the procedures provided in Subsection 7.2.3. It was found that the COV of θ_1 (corresponding to $\gamma_{t_{WD}}$ term) was highly negative (i.e., -1.6785). Hence, the $\gamma_{t_{WD}}$ term was removed from the full model to form the reduced model. Table 7-2 provides the MAPE and posterior statistics of the reduced $US_{WD,NV}$ model.

Table 7-2. MAPE and Posterior Statistics of the $US_{WD,NV}$ Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i	
						θ_0	θ_2
$US_{WD,NV}$	1.22	θ_0	1.0105	0.0022	0.002	1	
		θ_2	-1.6785	0.1362	-0.08	-0.47	1
		σ	0.0194	0.0073	0.38		

Table 7-2 shows that the coefficients (i.e., θ_i) of the predictor parameters have reasonably small COVs (i.e., 0.002 and -0.08). Also, the MAPE and σ are reasonably small (i.e., 1.22 percent and 0.0194, respectively). These observations indicate good prediction accuracies for the reduced model and show that the reduced model is the most parsimonious model. Hence, the reduced model in Table 7-2 will be referred to as Model $US_{WD,NV}$ herein.

The Model $US_{WD,NV}$ indicates that only the intercept and the two-way interaction terms are needed to predict the capacity when the tendons are fully grouted. This can be physically supported by the fact that if the chloride exposure level is low, it takes a long time for the

available chloride ions to get transported through the 0.25-inch grout cover, reach the strands in sufficient quantity, and initiate corrosion. So, the test durations (i.e., 12 and 21 months) were not long enough to cause chloride-induced corrosion, especially when the $\%sCl^-$ level of the test solution was 0.006 or 0.018. However, strands exposed to 0.18 and 1.8 $\%sCl^-$ exhibited relatively larger reduction in capacity.

A typical validation plot and its characteristics were provided in Subsection 7.2.3.5. Figure 7-7 shows the validation plot for the Model US_{WD, NV}. The spread (with an approximately equal width) of the most data points along the 1:1 line and within the $\pm\sigma$ region indicate an approximate agreement with both the homoskedasticity and normality assumptions, respectively.

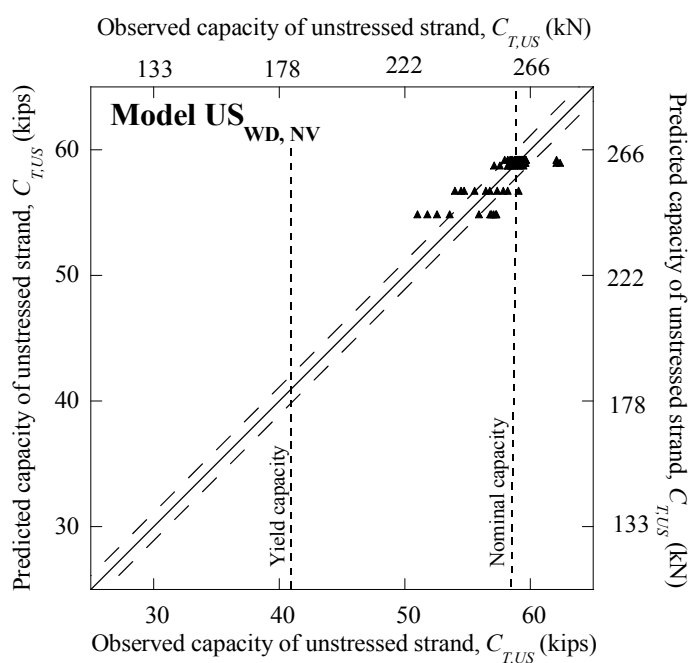


Figure 7-7. Validation Plot for the Model US_{WD, NV}.

7.5.2.6. Model assessment - Parallel Void conditions

Table 7-3 summarizes the MAPE and posterior statistics of the model form for PV conditions (provided in Equation (7.13)). These were assessed using the procedures provided in Subsection 7.2.3 and the 100 experimental data points.

Table 7-3. MAPE and Posterior Statistics of the US_{WD, PV} Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i		
						θ_0	θ_1	θ_2
US _{WD, PV}	1.61	θ_0	1.0232	0.0047	0.005	1		
		θ_1	-0.1553	0.0141	-0.09	-0.45	1	
		θ_2	-0.0153	0.0019	-0.12	0.01	0.84	1
		f	0.0256	0.0098	0.38			

Table 7-3 shows that the model parameters (i.e., θ_0 , θ_1 , and θ_2) have smaller COVs (i.e., 0.005, -0.09, and -0.12, respectively), indicating good confidence in the mean parameter estimations. The mean estimates of these parameters indicate that $C_{T,US}$, when exposed to PV and WD conditions, is more sensitive to the $\gamma_{t_{WD}}$ term than the $\ln[\gamma_{\%CT-}]\gamma_{t_{WD}}$ term. Table 7-3 also shows that MAPE (i.e., 1.61 percent) and σ (i.e., 0.0256) of the Model US_{WD, PV} are reasonably small. Because of these reasons, the model in Table 7-3 is referred to as Model US_{WD, PV} herein.

A typical validation plot and its characteristics were provided in Subsection 7.2.3.5. The validation plot for the Model US_{WD, PV} is shown in Figure 7-8. Among the 100 data points, the four data points near the top right region show underestimation and the two data points near the bottom left region show overestimation of $C_{T,US}$. Most of the remaining data points fall along the 1:1 line (spread with an approximately equal width) and exhibit no significant bias or residual trend, indicating a reasonably good agreement with the homoskedasticity assumption. The spread of the most data points along the 1:1 line indicates good prediction accuracy. In addition, most data points fall within and some fall outside the $\pm\sigma$ region, indicating reasonably good agreement with the normality assumptions.

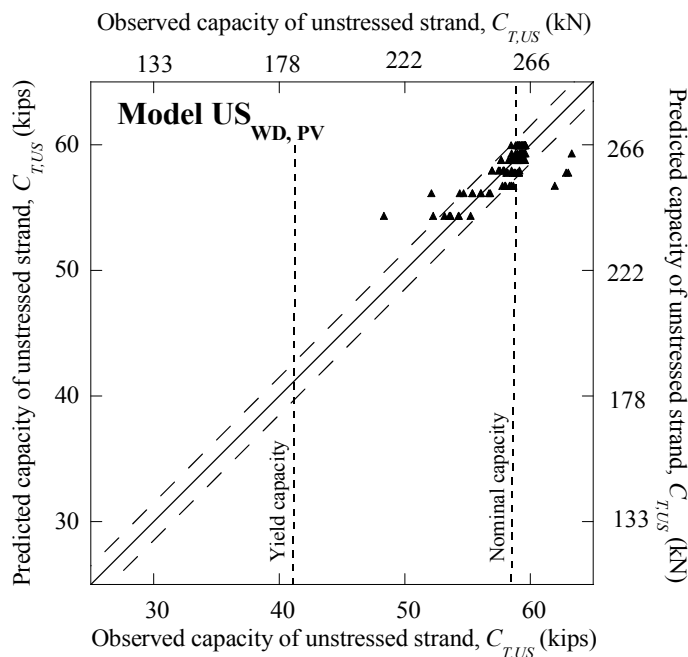


Figure 7-8. Validation Plot for the Model US_{WD, PV}.

7.5.2.7. Model assessment – Bleedwater, Inclined, and Orthogonal Void conditions

Table 7-4 provides the MAPE and posterior statistics for the full model form for BIOV conditions (provided in Equation (7.13)). The model was assessed using the 239 data points.

Table 7-4. MAPE and Posterior Statistics of the US_{WD, BIOV} Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i		
						θ_0	θ_1	θ_2
US _{WD, BIOV}	3.15	θ_0	1.0333	0.0056	0.005	1		
		θ_1	-0.3567	0.0124	0.03	-0.61	1	
		θ_2	-0.0285	0.0015	-0.05	0.01	0.74	1
		σ	0.0350	0.0107	0.31			

Table 7-4 shows that the MAPE and σ for the full model are reasonably small (i.e., 3.15 percent and 0.0350, respectively) indicating good prediction accuracy. The mean estimates

for θ_1 and θ_2 indicate that the capacity is more sensitive to $\gamma_{t_{WD}}$ than $\ln[\gamma_{\%sCl^-}] \gamma_{t_{WD}}$. The high confidence levels on the mean estimates of the model parameters θ_0 , θ_1 , and θ_2 (i.e., 1.0333, -0.3567 , and -0.0285 , respectively) are evident from their reasonably small COVs (i.e., 0.005, -0.03 , -0.05 , respectively). Hence, the reduced model in Table 7-4 is referred to as Model US_{WD, BIOV} herein.

A typical validation plot and its characteristics were provided in Subsection 7.2.3.5. In the validation plot in Figure 7-9, most data points lie within the $\pm\sigma$ region. This exhibits reasonably good prediction accuracy. As shown in Figure 7-9, although variability exists among the observed $C_{T,US}$ for identical exposure conditions, the mean of the observed values of $C_{T,US}$ corresponding to a particular predicted $C_{T,US}$ lies very close to the 1:1 line. This indicates good accuracy in model prediction, given the inherent variability in the corrosion process.

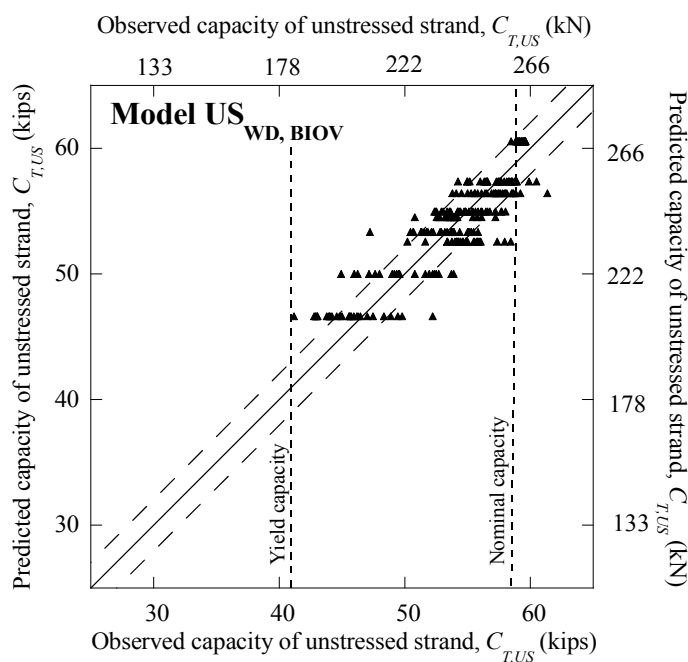


Figure 7-9. Validation Plot for the Model US_{WD, BIOV}.

7.5.2.8. Prediction of tension capacity of unstressed strands

In this subsection, the US_{WD} models are used to predict or extrapolate $C_{T,US}$ when various void, moisture, and chloride conditions exist. Figure 7-10(a), (b), (c), and (d) show the extrapolated

$C_{T,US}$ under WD exposure conditions with 0.006, 0.018, 0.18, and 1.8 % sCl^- solutions, respectively. These predictions are based on the assumption that ϕ_{wet} is equal to 0.17 (see Equation (7.11) in Subsections 7.4.1 and 7.4.3 for details).

A comparison between Figure 7-10(a) and Figure 7-10(b), (c), or (d) shows that when voids exist, $C_{T,US}$ can drop below $MUTS_{ARS}$ within approximately 1 year. However, the $C_{T,US}$ remained higher than yield capacity for up to approximately 18, 14, 10, and 7 years, respectively, for the WD conditions with 0.006, 0.018, 0.18, and 1.8 % sCl^- , respectively. These times are shown by the vertical dashed arrows in Figure 7-10. The possibility of capacity to drop below $MUTS_{ARS}$ within a year, especially for conditions where WD cycles can be numerous, shows that the frequency of inspection and structural assessment of PT bridges should be performed on a regular basis. The inspection frequency should depend on the likelihood of infiltration of water, chlorides, or both into the ducts. Such studies are not performed as part of this study.

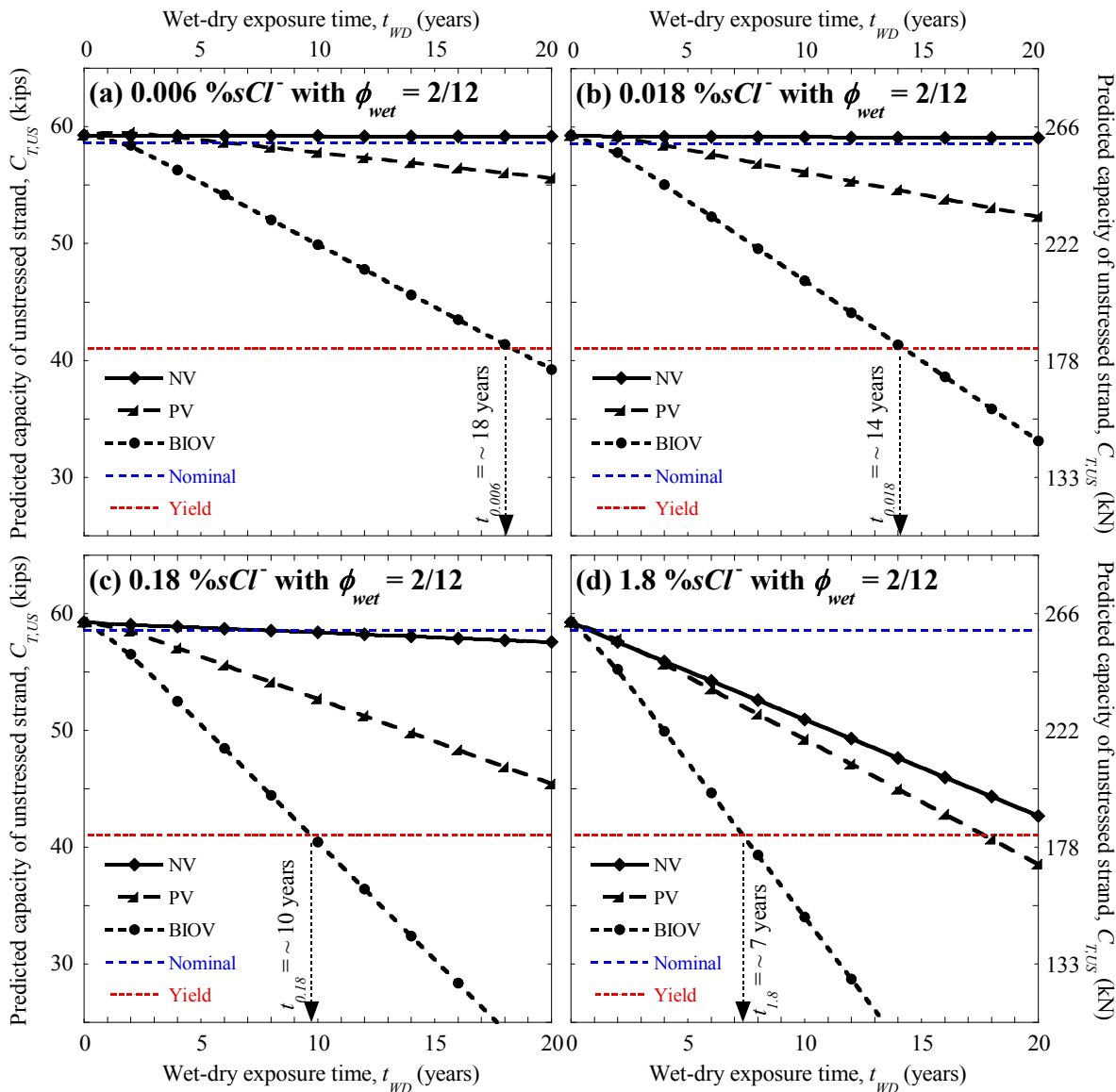


Figure 7-10. Capacity of Unstressed Strands under WD Exposure to Various Chloride Solutions (Predicted Using US_{WD} Models).

7.5.3. STEP WD-3: Models for stressed strands under WD exposure conditions (using stressed strand data only)

In this subsection, three probabilistic models are developed to predict C_T of stressed strands exposed to WD conditions along with NV, PV, and BIOV conditions. These models are denoted as Model $SS_{WD, NV}$, Model $SS_{WD, PV}$, and Model $SS_{WD, BIOV}$, respectively. Experimental data from 60, 85, and 79 stressed strand samples have been used to develop the $SS_{WD, NV}$, $SS_{WD, PV}$, and $SS_{WD, BIOV}$, respectively. Prior to formulating the models, the effects of various test parameters on $C_{T,SS}$ are diagnosed.

7.5.3.1. *Diagnosis of the effect of wet-dry exposure time*

The stressed strand samples were tested at 0, 12, 16, and 21 months. Figure 7-11(a), (b), and (c) indicate that capacity of stressed strands under NV, PV, and BIOV conditions can also be modeled using a linear function of t_{WD} as used in the case of unstressed strands. A comparison between Figure 7-11(a), (b), and (c) shows that the slope of this linear fit increases as the void condition changes from NV, to PV, to BIOV. However, in general, the scatter in the capacity data increases as t_{WD} increases. Hence, an interaction term, as presented in Subsection 7.5.3.3, can help in minimizing the heteroskedasticity and improving the model quality.

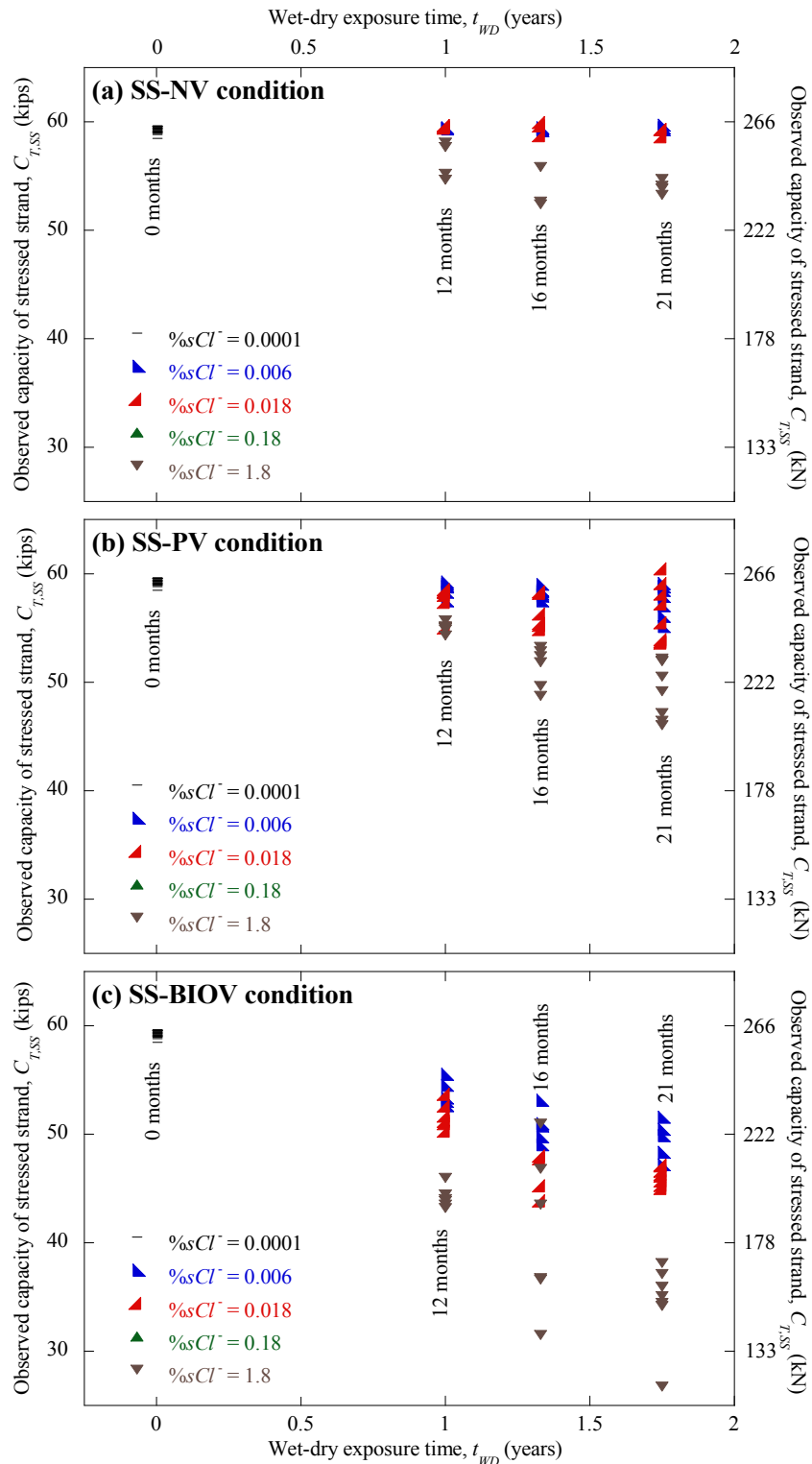


Figure 7-11. Effect of Wet-Dry Exposure Time on Capacity of Stressed Strands (with $\phi_{wet} = 0.5$).

7.5.3.2. *Diagnosis of the effect of chloride concentration in exposure solution*

Figure 7-12(a) and Figure 7-13(a) show the scatter plots between $C_{T,SS}$ (for NV conditions) and $\%sCl^-$ and $\ln[\%sCl^-]$, respectively. A comparison between these diagnostic plots indicates that $\%sCl^-$ is a more appropriate capacity predictor term than $\ln[\%sCl^-]$ when NV conditions exist. Similar to the observation made in the case of unstressed strands (see Figure 7-6[a]), negligible C_T loss is observed until 0.018 $\%sCl^-$, after which there is a slight increase in the slope. This is probably because longer time was needed to build up sufficient amount of chlorides at the surface of completely embedded strands and then initiate/propagate corrosion, when exposed to 0.006 and 0.018 $\%sCl^-$ solutions than when exposed to 1.8 $\%sCl^-$ solution. The comparison between Figure 7-12(b) and Figure 7-13(b) for PV conditions indicates that both $\%sCl^-$ and $\ln[\%sCl^-]$ may be used as a linear term to predict $C_{T,SS}$. However, stressed strands were not tested at 0.18 $\%sCl^-$ condition or any other intermediate $\%sCl^-$ levels. Hence, the acceptance of $\%sCl^-$ term as a predictor can be questionable. In addition, Figure 7-13(b) indicates an approximate linear relationship between capacity and $\ln[\%sCl^-]$. Because of these reasons, and in order to keep the same model form as in the case of the $US_{WD, PV}$ model, the $\ln[\%sCl^-]$ term seems to be a more appropriate predictor term than the $\%sCl^-$ term. Figure 7-12(c) for BIOV conditions indicates a non-linear relationship between $\%sCl^-$ level and capacity. In addition, Figure 7-13(c) clearly shows a good linear relationship between $\ln[\%sCl^-]$ and $C_{T,SS}$ for the range of $\%sCl^-$ tested. Hence, for stressed strands under BIOV conditions, $\ln[\%sCl^-]$ is a more appropriate capacity predictor than $\%sCl^-$. As observed with the case of unstressed strands in Subsection 7.5.2.2, Figure 7-12 and Figure 7-13 indicate a general trend of increasing scatter as the $\%sCl^-$ increases. This might induce some heteroskedasticity into the model. A two-way interaction term between t_{WD} and $\%sCl^-$ or $\ln[\%sCl^-]$ might help in making the models more homoskedastic.

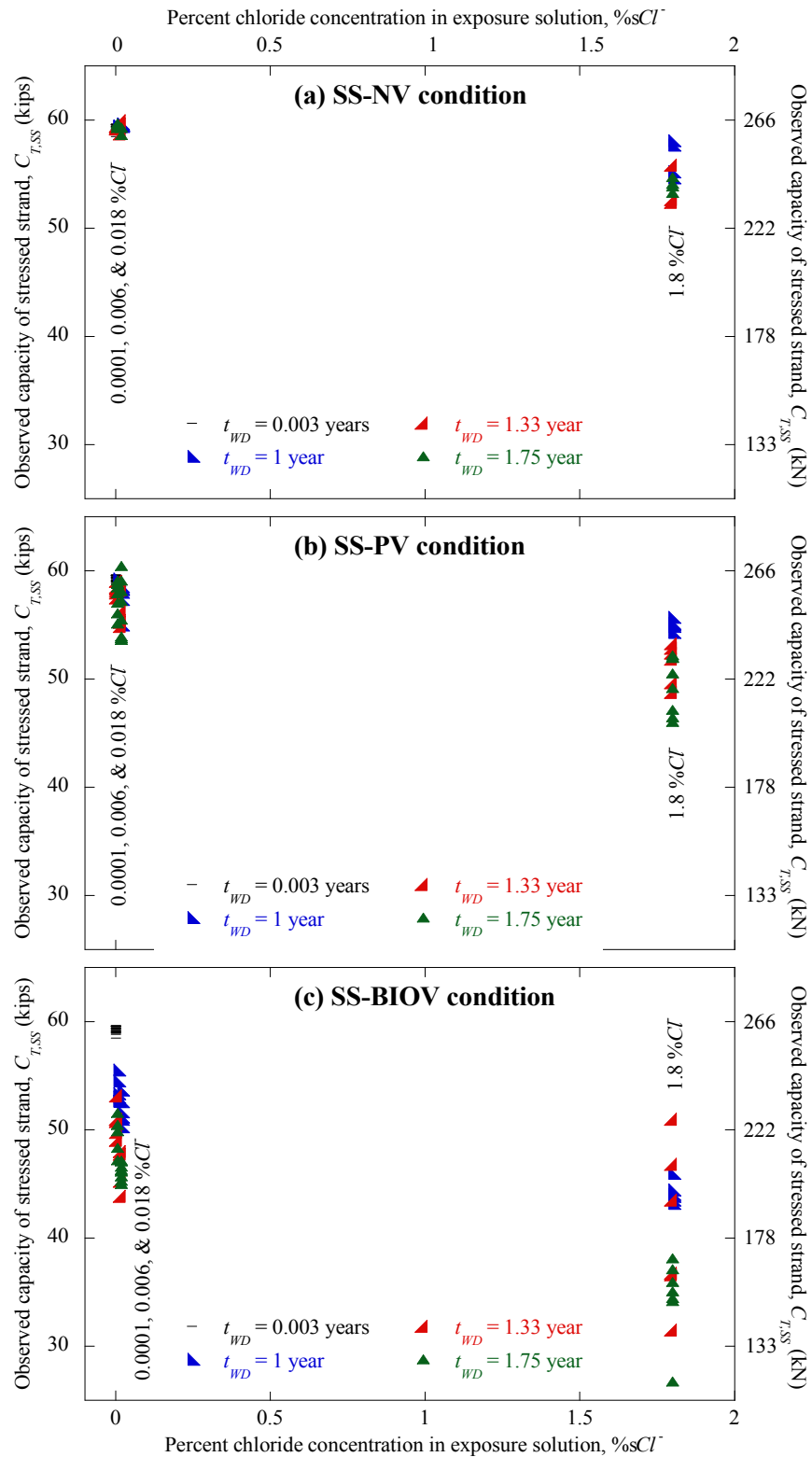


Figure 7-12. Effect of %sCl⁻ on Capacity of Stressed Strands (with $\phi_{wet} = 0.5$).

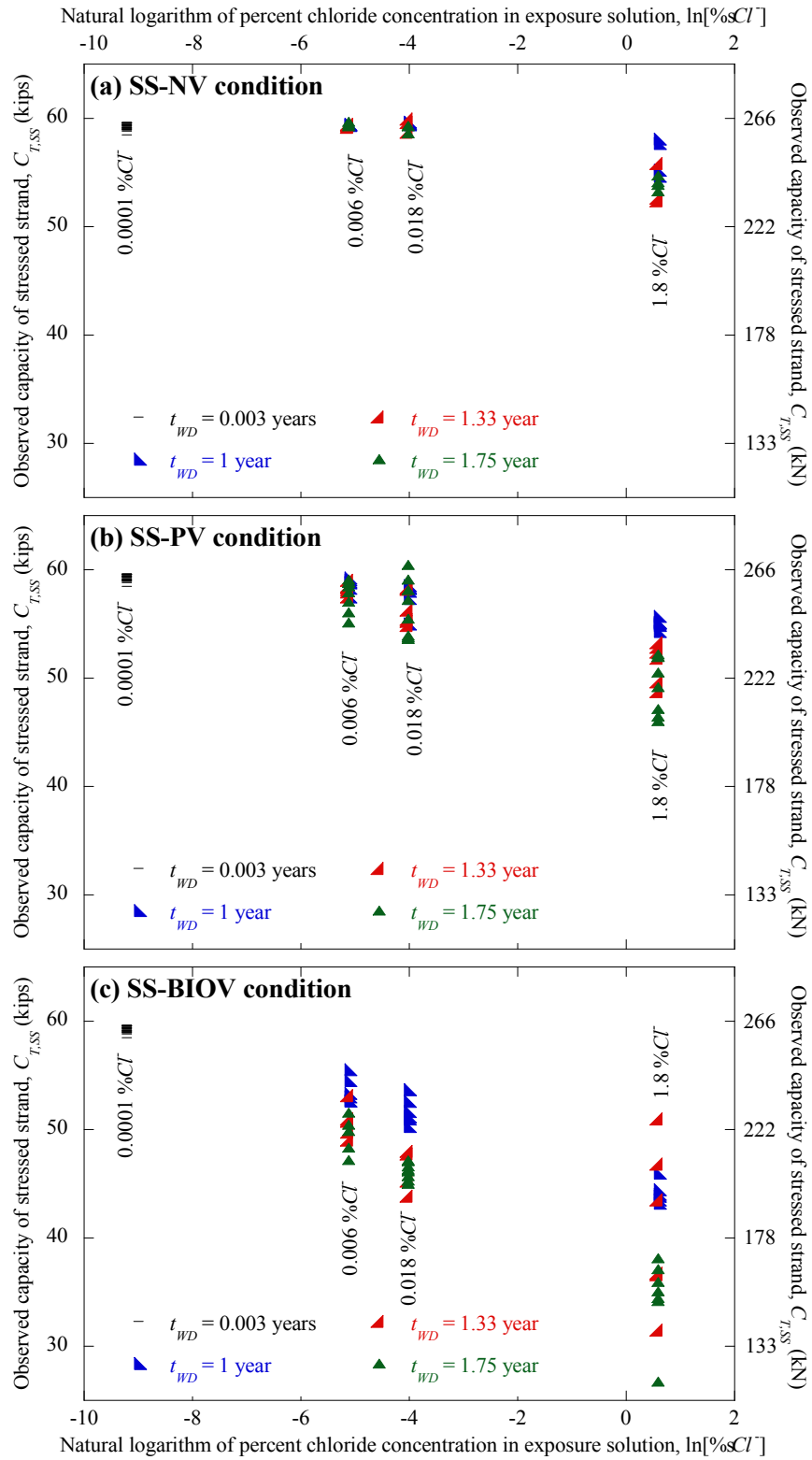


Figure 7-13. Effect of $\ln[\%sCl]$ on Capacity of Stressed Strands (with $\phi_{wet} = 0.5$).

7.5.3.3. *Diagnosis of the effect of interaction between chloride and time parameters*

In the previous two subsections, a need for an interaction term to improve prediction accuracy and agreements with homoskedasticity assumptions was cited. Detailed justifications for incorporating a two-way interaction term was provided in Subsection 7.5.2.3 on modeling $C_{T,US}$. The same justifications are valid for modeling $C_{T,SS}$. In the previous subsection, it was found that $\%sCl^-$ is an appropriate capacity predictor term for NV conditions. Hence, $\%sCl^- \times t_{WD}$ is an appropriate two-way interaction term to predict $C_{T,SS}$ under NV conditions. In the previous subsection, it was also found that the $\ln[\%sCl^-]$ is an appropriate predictor for $C_{T,SS}$ under PV and BIOV conditions. Hence, $\ln[\%sCl^-] t_{WD}$ can be used as an appropriate two-way interaction term to predict $C_{T,SS}$ under PV and BIOV conditions.

7.5.3.4. *Model formulation*

Based on the diagnostic study and physical principles behind the time-dependent corrosion process, three probabilistic model forms to predict $C_{T,SS}$ under NV, PV, and BIOV conditions are formulated as follows:

$$\begin{aligned} R_{C_{T,SS}}(\mathbf{x}, \Theta) &= \theta_0 + \theta_1 \gamma_{t_{WD}} + \theta_2 \gamma_{\%sCl^-} \gamma_{t_{WD}} + \sigma \varepsilon \quad (\text{for NV condition}) \\ R_{C_{T,SS}}(\mathbf{x}, \Theta) &= \theta_0 + \theta_1 \gamma_{t_{WD}} + \theta_2 \ln[\gamma_{\%sCl^-}] \gamma_{t_{WD}} + \sigma \varepsilon \quad (\text{for PV and BIOV conditions}) \end{aligned} \quad (7.14)$$

where the terms are as defined in Equation (7.1) and Subsection 7.4.1. These SS_{WD} model forms are similar to the US_{WD} model forms (i.e., Equation (7.13)). When the t_{WD} is 0, the intercept term will predict $C_{T,SS}$ that is practically very close to $MUTS_{ARS}$. The term $\gamma_{t_{WD}}$ in the predictor parameters will capture the effect of t_{WD} , whereas the two-way interaction term (i.e., $\gamma_{\%sCl^-} \gamma_{t_{WD}}$ or $\ln[\gamma_{\%sCl^-}] \gamma_{t_{WD}}$) between the chloride and time parameters will minimize the model heteroskedasticity. The parameters in these model forms are assessed using the procedures given in Subsection 7.2 and are provided in the next three subsections.

7.5.3.5. Model assessment – No Void conditions

A total of 60 data points were used to develop this model. MAPE and posterior statistics of the full model form for NV conditions (see Equation (7.14)) was assessed using the procedures given in Subsection 7.2.3. However, because the COV of θ_1 (i.e., the model parameter corresponding to $\gamma_{t_{WD}}$) is as large as 1.77, it seems that the full model does not correspond to the most parsimonious parameter set. Because of this, the $\gamma_{t_{WD}}$ term was removed to formulate a reduced model for NV conditions. Table 7-5 summarizes the MAPE and posterior statistics of this reduced model. The reasonably small COVs of coefficients (i.e., θ_i) in the reduced model indicate that the model can predict $C_{T,SS}$ with reasonable accuracy. The MAPE of the reduced model is 0.73 percent. In addition, the model error parameter, σ , of the reduced model is 0.0121. Further reduction of the model resulted in large MAPE and model errors. Based on these findings, the reduced model in Table 7-5 is referred to as Model $SS_{WD, NV}$ herein.

Table 7-5. MAPE and Posterior Statistics of the $SS_{WD, NV}$ Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i	
						θ_0	θ_2
$SS_{WD, NV}$	0.73	θ_0	1.0126	0.0017	0.002	1	
		θ_2	-2.2542	0.1141	-0.05	-0.43	1
		σ	0.0121	0.0053	0.44		

Based on the discussions in Subsection 7.2.3.5 on validation plots, Figure 7-14 indicates a reasonably good prediction model. The spread of the data in Figure 7-14 has an approximately equal width, lies along the 1:1 line, and does not show any significant residual trend or bias in prediction, indicating good agreement with the homoskedasticity assumption. Also, the most data points lie inside the $\pm\sigma$ region, indicating that normality assumption is reasonably agreeable. Because of the relatively small amount of corrosion and less scatter among the capacity data, the Model $SS_{WD, NV}$ has less MAPE than the models for PV and BIOV conditions discussed next.

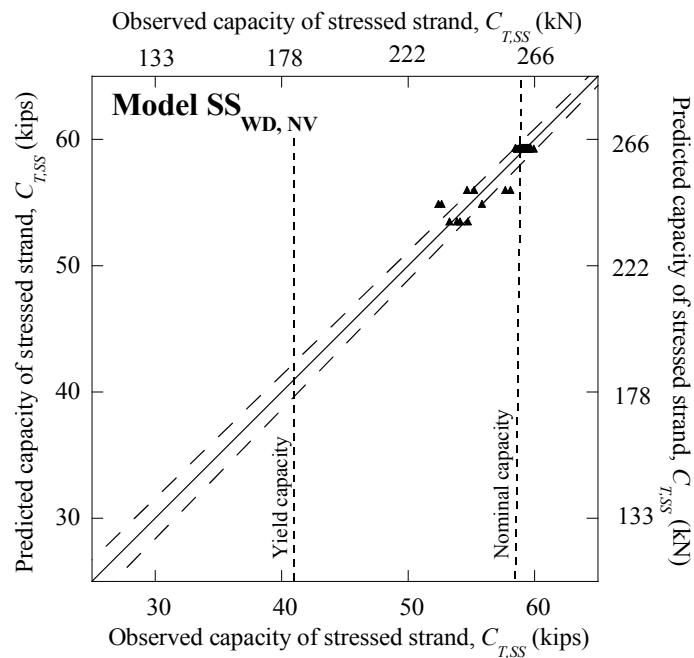


Figure 7-14. Validation Plot for the Model SS_{WD, NV}.

7.5.3.6. Model assessment – Parallel Void conditions

The model parameters for the SS_{WD, PV} model were assessed using the 85 data points. These statistical results are summarized in Table 7-6. The MAPE and σ (i.e., 1.89 percent and 0.0249, respectively) of the full model are reasonably small. In addition, the absolute COVs of the model parameters corresponding to the predictor parameters are less than 0.07, indicating good confidence in the mean estimates. In summary, the full model seems to be the most parsimonious model and has good prediction accuracy. Hence, the full model in Table 7-6 is referred to as Model SS_{WD, PV} herein.

Table 7-6. MAPE and Posterior Statistics of the $SS_{WD, PV}$ Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i		
						θ_0	θ_2	θ_3
$SS_{WD, PV}$	1.89	θ_0	1.0129	0.0048	0.005	1		
		θ_1	-0.2719	0.0141	-0.05	-0.46	1	
		θ_2	-0.0277	0.0018	-0.07	0.00	0.82	1
		σ	0.0249	0.0100	0.40			

Based on the discussions in Subsection 7.2.3.5 on validation plots, the validation plot in Figure 7-15 suggests that the Model $SS_{WD, PV}$ provides good prediction. In Figure 7-15, the width of the spread of the data is approximately equal, indicating good agreement with the homoskedasticity assumption. Also, Figure 7-15 indicates no specific significant residual trend or bias. The means of the observed capacities corresponding to the predicted capacities seem to lie along the 1:1 line. This indicates reasonably good prediction of capacity, given the scatter in the data among the samples with identical exposure conditions. Also, the normality assumption is approximately satisfied because most data points lie inside and some lie outside the $\pm\sigma$ region.

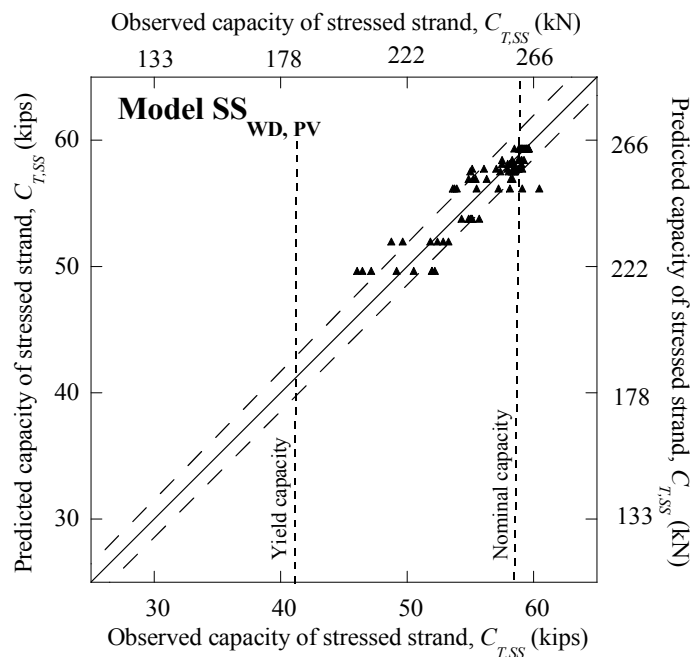


Figure 7-15. Validation Plot for the Model SS_{WD, PV}.

7.5.3.7. Model assessment – Bleedwater, Inclined, and Orthogonal Void conditions

The model form for BIOV conditions in Equation (7.14) is assessed using the 79 data points from the stressed strands exposed to OV conditions only. However, this model can be used to predict $C_{T,SS}$ under BIOV conditions and is denoted as the SS_{WD, BIOV} model. This is justified because $C_{T,SS}$ under OV, IV, and BV conditions are statistically similar (see Subsection 6.5.5 for details). Table 7-7 provides MAPE and posterior statistics for the SS_{WD, BIOV} model. Table 7-7 shows that the MAPE of the full model form is 3.25 percent. Although reasonable, this is slightly larger than that observed for the SS_{WD, PV} model and is because of the larger scatter in the observed capacity data. Table 7-7 shows that the σ of the full model is 0.0175 (a reasonably small value). In addition, COVs of the model parameters (i.e., θ_0 , θ_1 , and θ_2) corresponding to the intercept $\gamma_{t_{WD}}$ and $\ln[\gamma_{\%Cl^-}] \gamma_{t_{WD}}$ terms are reasonably small (i.e., 0.01, -0.04, -0.07, respectively). Hence, the full model in Table 7-7 is referred to as Model SS_{WD, BIOV} herein.

Table 7-7. MAPE and Posterior Statistics of the $SS_{WD, BIOV}$ Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i		
						θ_0	θ_1	θ_2
$SS_{WD, BIOV}$	3.25	θ_0	1.0095	0.0083	0.01	1		
		θ_1	-0.6274	0.0250	-0.04	-0.45	1	
		θ_2	-0.0485	0.0033	-0.07	0.01	0.83	1
		σ	0.0427	0.0175	0.41			

Discussions on reading the validation plots were provided earlier in Subsection 7.2.3.5. Figure 7-16 shows the validation plot for the Model $SS_{WD, BIOV}$. In this validation plot, the means of the observed values of $C_{T,SS}$ corresponding to a particular predicted $C_{T,SS}$ are falling near the 1:1 line. In Figure 7-16, the large scatter among the data points corresponding to the predicted $C_{T,SS}$ of approximately 40 kips (178 kN) may seem to suggest that the homoskedasticity assumption is not satisfied. However, this is the inherent scatter in $C_{T,SS}$ that is observed when exposed to 0.18 % sCI for a period of 16 months and is not generated due to the model inadequacy. Moreover, $\sigma \times MUTS_{ARS}$ is equal to 3.02 kips (13 kN), which is less than the standard deviation of the observed $C_{T,SS}$ (i.e., 7.32 kips [32.6 kN]). Hence, the reduced model can be assumed to be practically homoskedastic. Also, because most data points lie within and some lie outside the $\pm\sigma$ region, the normality assumption seems to be reasonably validated.

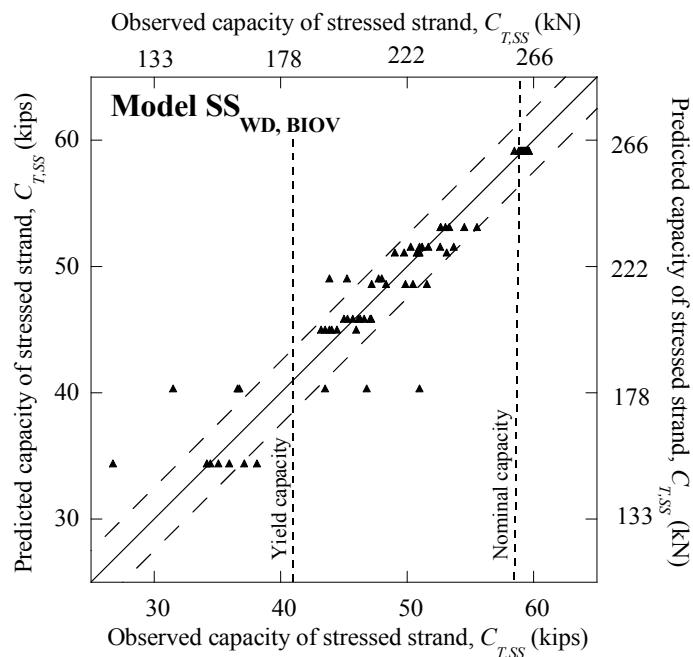


Figure 7-16. Validation Plot for the Model SS_{WD, BIOV}.

7.5.3.8. Prediction of tension capacity of stressed strands

Subsection 7.4.3 presented $C_{T,US}$ extrapolated using the US_{WD} models. This subsection presents $C_{T,SS}$ extrapolated or predicted using the SS_{WD} models. Figure 7-17(a), (b), (c), and (d) show the extrapolated $C_{T,SS}$ values when exposed to WD conditions with 0.006, 0.018, 0.18, and 1.8 % sCI^- solutions, respectively. These predictions are based on the assumption that ϕ_{wet} is equal to 0.17 (see Equation (7.11) in Subsection 7.4.1 and 7.4.3 for details). Note that ϕ_{wet} could be larger or smaller than this value, depending on varying field conditions. The larger the ϕ_{wet} , the faster the rate of corrosion and capacity loss will be, and vice versa.

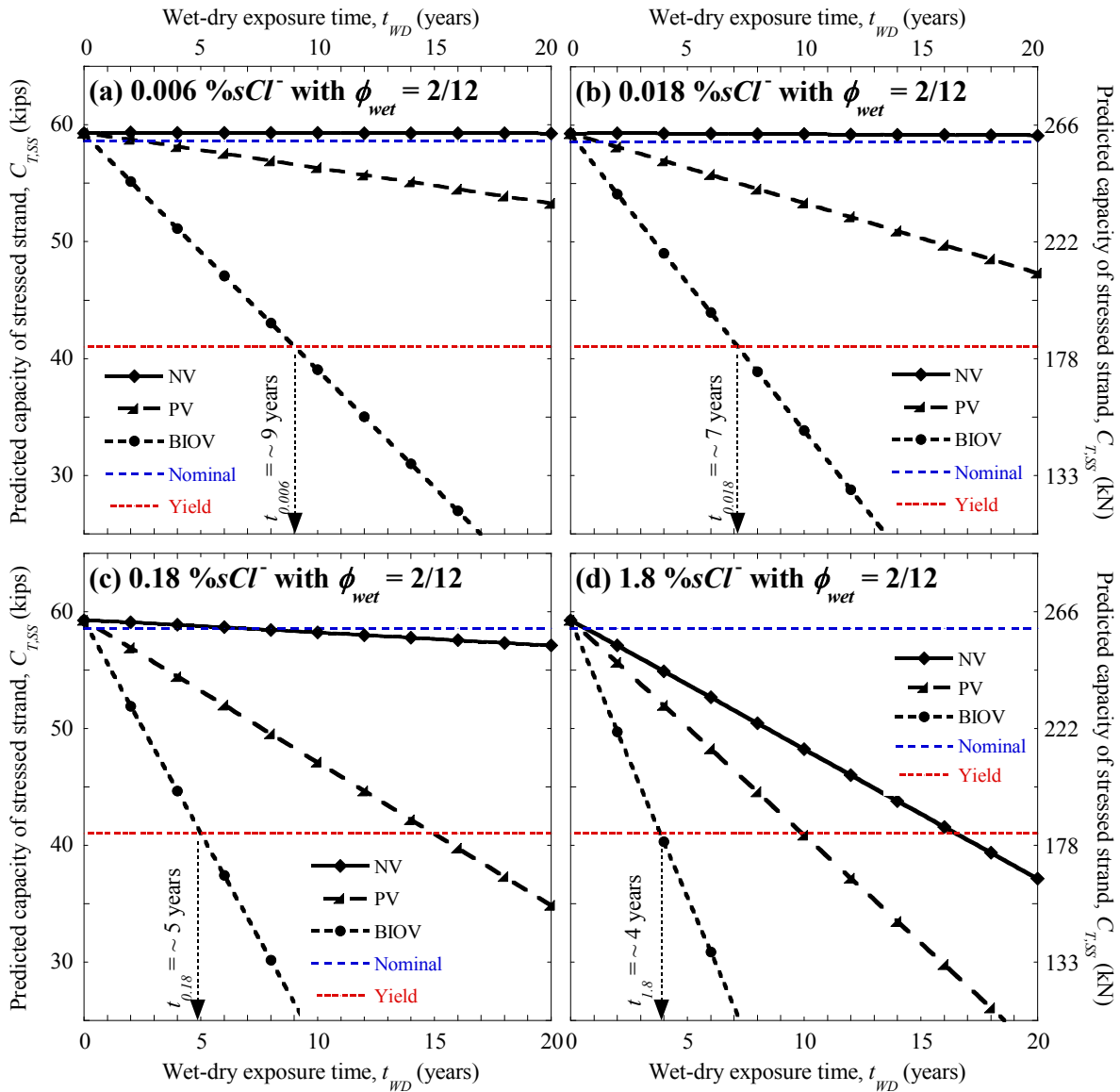


Figure 7-17. Capacity of Stressed Strands under WD Exposure ($\phi_{wet} = 0.17$) to Various Chloride Solutions (Predicted Using SS_{WD} Models).

Figure 7-17(a) and (b) show that negligible reduction in $C_{T,SS}$ is expected when exposed to NV and 0.006 or 0.18 %sCl⁻ conditions for t_{WD} equal to 20 years. Figure 7-17(c) and (d) show that, in the case of NV conditions, the exposure to 0.18 and 1.8 %sCl⁻ solutions (i.e., moderate and high chloride conditions) can cause C_T to drop below $MUTS_{ARS}$ (or nominal capacity) within 7 and 1 year, respectively. From Figure 7-17(a), (b), (c), and (d), it can be concluded that, for PV and BIOV conditions, the predicted $C_{T,SS}$ can, in general, fall below

$MUTS_{ARS}$ (or nominal capacity) within approximately 1 year. For PV samples exposed to 0.006 %s Cl^- condition, $C_{T,SS}$ dropped below $MUTS_{ARS}$ within approximately 2 years. Figure 7-17(a), (b), (c), and (d) show that when BIOV and WD conditions exist, $C_{T,SS}$ can drop to a value below yield capacity within approximately 9, 7, 5, and 4 years (shown by vertical dashed arrows) when exposed to 0.006, 0.018, 0.18, and 1.8 %s Cl^- solutions, respectively.

7.5.4. STEP WD-4: Models for stressed strands under WD exposure conditions (using unstressed strand model and stressed strand data)

This section presents three probabilistic capacity models to predict $C_{T,SS}$ under WD exposure conditions. These models are denoted as Model US-SS_{WD, NV}, Model US-SS_{WD, PV}, and Model US-SS_{WD, BIOV}. Experimental data obtained from 60, 85, and 79 stressed strand samples and $C_{T,US}$ values predicted using US_{WD, NV}, US_{WD, PV}, and US_{WD, BIOV} models have been used to develop the US-SS_{WD, NV}, US-SS_{WD, PV}, and US-SS_{WD, BIOV} models, respectively.

7.5.4.1. Diagnosis of the relationship between the capacities of unstressed and stressed strands

Figure 7-18(a), (b), and (c) show the scatter plots between the predicted $C_{T,US}$ and observed $C_{T,SS}$ under similar exposure conditions. At first, it might seem that a linear relationship between $C_{T,US}$ and $C_{T,SS}$ is reasonable. However, preliminary model assessments indicated that a linear model would result in the overprediction of $C_{T,SS}$ near $MUTS_{ARS}$. This is because the slope of the relationship between $C_{T,US}$ and $C_{T,SS}$ reduces as $C_{T,SS}$ approaches $MUTS_{ARS}$. Hence, a power model seems to be more appropriate than a linear model.

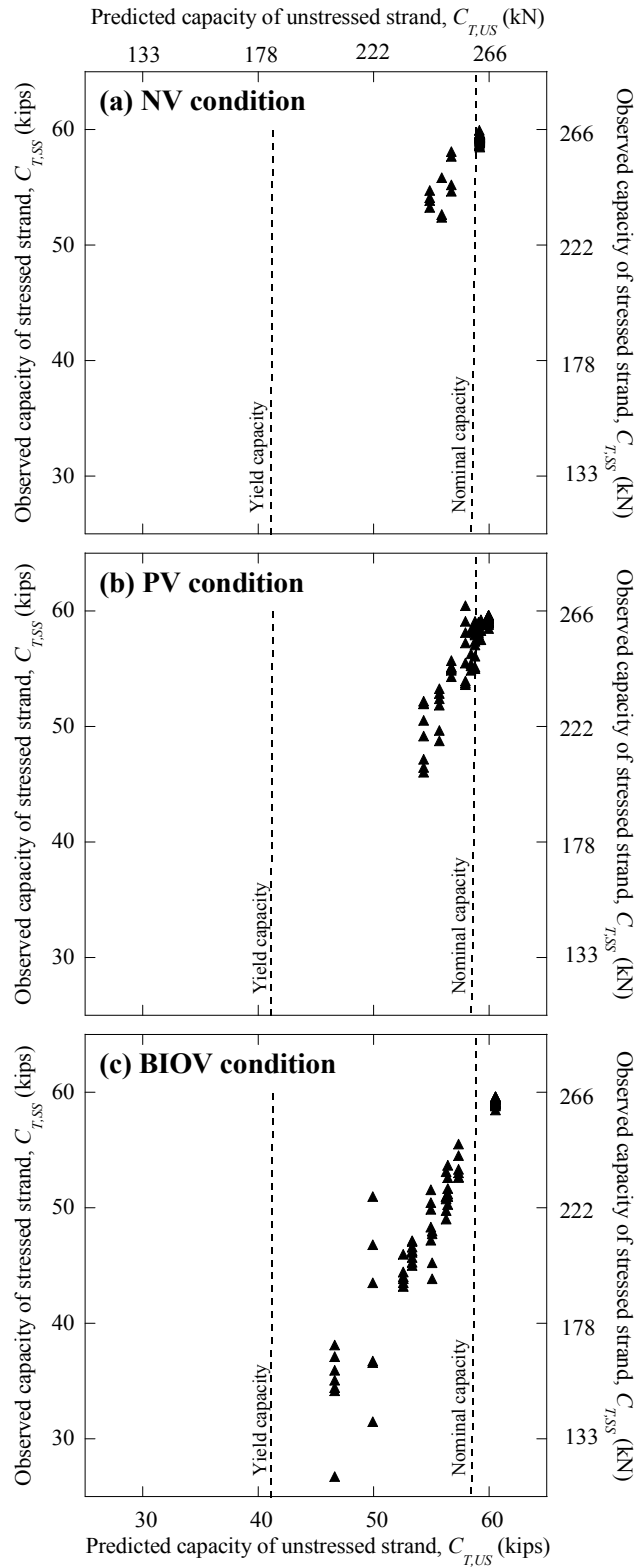


Figure 7-18. Scatter Plots between the Capacities of Unstressed and Stressed Strands.

7.5.4.2. Model formulation

As determined in the previous subsection, a power model is formulated as follows:

$$R_{C_{T,SS}} = \theta_0 \left(\gamma_{C_{T,US}} \right)^{\theta_1} + \sigma \varepsilon \quad (\text{for NV, PV, and BIOV conditions}) \quad (7.15)$$

where the terms are as defined in Subsection 7.4.1 and $\gamma_{C_{T,US}}$ is equal to $R_{C_{T,US}}$ predicted using $C_{T,US}$, which is the median value predicted using an appropriate US_{WD} model. In other words, $\gamma_{C_{T,US}}$ is the expression on the right side of the US_{WD} models after excluding the model error term $\sigma \varepsilon$. The power model form in Equation (7.15) does not have an intercept term and, hence, will ensure that $C_{T,SS}$ is 0 when $C_{T,US}$ is 0. Also, the power model form helps in minimizing the overprediction of $C_{T,SS}$ as it approaches $MUTS_{ARS}$. The model parameters are assessed using the maximum likelihood approach for non-linear models given in Subsection 7.2.3. Following is a discussion on the assessment of $US-SS_{WD, NV}$, $US-SS_{WD, PV}$, and $US-SS_{WD, BIOV}$ models.

7.5.4.3. Model assessment – No Void conditions

This model can be used to predict C_T of stressed strands inside fully grouted tendons. This model is assessed using the 60 observed data on $C_{T,SS}$ and the corresponding values of $C_{T,US}$ (predicted using the $US_{WD, NV}$ model). Table 7-8 shows that the COVs (i.e., 0.001 and 0.05) of θ_0 and θ_1 are reasonably small, indicating good confidence in their mean estimates. Also, the mean and COV of the model error term (i.e., σ) are 0.0117 and 0.09, respectively. These results indicate that the $US-SS_{WD, NV}$ model can predict $C_{T,SS}$ with reasonable accuracy. The MAPE of the $US-SS_{WD, NV}$ model is 0.73 percent (equivalent to that of the $SS_{WD, NV}$ model). Based on these findings, the model in Table 7-8 is referred to as Model $US-SS_{WD, NV}$ herein.

Figure 7-19 indicates that $US-SS_{WD, NV}$ can provide a reasonably good prediction of capacity. The data in Figure 7-19 are spread along the 1:1 line, with an approximately equal width, and do not show any significant residual trend. These indicate reasonably good

agreement with the homoskedasticity assumption. Also, the most data points lie inside and some data points lie outside the $\pm\sigma$ region, indicating that normality assumption is reasonably valid.

Table 7-8. MAPE and Posterior Statistics of the US-SS_{WD, NV} Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i		
						θ_0	θ_1	σ
US-SS _{WD, NV}	0.73	θ_0	0.9983	0.0014	0.001	1		
		θ_1	1.3576	0.0648	0.05	0.04	1	
		\square	0.0117	0.0011	0.09	0.06	-0.01	1

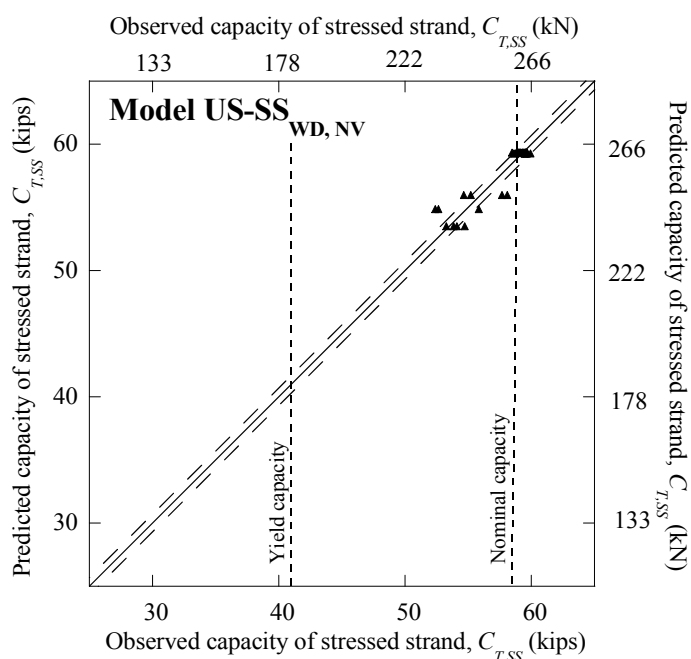


Figure 7-19. Validation Plot for US-SS_{WD, NV} Model.

7.5.4.4. Model assessment – Parallel Void conditions

Model US-SS_{WD, PV} can be used to predict $C_{T,SS}$ when voids exist at or near the midspan regions in tendons. The model parameters for the model form for PV conditions (provided in Equation (7.15)) were assessed using the 85 stressed strand data and the corresponding $C_{T,US}$ values predicted using the US_{WD, PV} model. The US_{WD, PV} model was assessed based on 100 unstressed strand data points. Hence, a total of 185 data points are effectively used in

developing the US-SS_{WD, PV} model. The statistical parameters of the US-SS_{WD, PV} model are summarized in Table 7-9.

Table 7-9. MAPE and Posterior Statistics of the US-SS_{WD, PV} Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i		
						θ_0	θ_1	θ_2
US-SS _{WD, PV}	1.93	θ_0	0.9748	0.0028	0.003	1		
		θ_1	1.8139	0.1009	0.06	0.14	1	
		σ	0.0244	0.0020	0.08	-0.04	0.002	1

Table 7-9 shows that the MAPE of the US-SS_{WD, PV} model is 1.93 percent. This is almost identical to the MAPE of the SS_{WD, PV} model (i.e., 1.89 percent). Table 7-9 also shows that the model error term (i.e., σ) of the US-SS_{WD, PV} model is 0.0244, which is a reasonably small value. In addition, COVs of the model parameters, θ_0 , and θ_1 , are 0.003 and 0.06, respectively. These are reasonably small and indicate good confidence in the corresponding mean estimates (i.e., 0.9748 and 1.8139, respectively). Because of these reasons, the full model in Table 7-9 is referred to as Model US-SS_{WD, PV} herein.

Figure 7-20 shows the validation plot for Model US-SS_{WD, PV} and looks very similar to the plot in Figure 7-15 for the SS_{WD, PV} model. In Figure 7-20, the means of the observed capacities corresponding to a particular predicted capacity are falling near the 1:1 line. Also, the data points are spread in an approximately equi-width band along the 1:1 line. Also, no residual trend pattern is observed among the data points in Figure 7-20. These indicate that the model can be assumed to be practically homoskedastic. Also, the normality assumption seems to be reasonably agreed because most data points lie within and some lie outside the $\pm\sigma$ region.

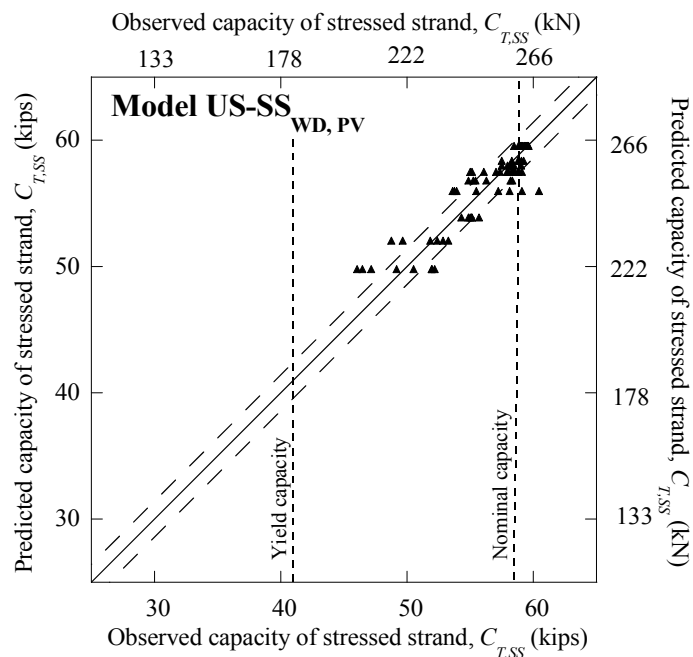


Figure 7-20. Validation Plot for US-SS_{WD, PV} Model.

7.5.4.5. Model assessment – Bleedwater, Inclined, and Orthogonal Void conditions

Model US-SS_{WD, BIOV} can be used to predict $C_{T,SS}$ when voids exist at or near the anchorage regions of PT columns or girders. The power model form for BIOV conditions (provided in Equation (7.15)) was assessed using the 79 stressed strand data points and the $C_{T,US}$ predicted using the US_{WD, BIOV} model. The US_{WD, BIOV} model was developed using 239 unstressed strand data points. Hence, a total of 318 data points are effectively used in developing the US-SS_{WD, BIOV} model. The MAPE and posterior statistics of the US-SS_{WD, BIOV} model are provided in Table 7-10.

Table 7-10. MAPE and Posterior Statistics of the US-SS_{WD, BIOV} Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i		
						θ_0	θ_1	θ_2
US-SS _{WD, BIOV}	3.15	θ_0	0.9463	0.0064	0.007	1		
		θ_1	2.0301	0.0773	0.04	0.47	1	
		σ	0.0411	0.0034	0.08	0.15	0.04	1

In Subsection 7.5.3.7, the MAPE of the $SS_{WD, BIOV}$ model was found to be 3.25 percent. Table 7-10 shows that the MAPE of the US-SS model is 3.15 percent. In addition to the reasonably small MAPE, the model error term (i.e., σ) of the US-SS model is 0.0411, indicating good model accuracy. The high confidence levels on the mean estimates of the model parameters (i.e., θ_0 and θ_1) are demonstrated by their small COVs (i.e., 0.007 and 0.04, respectively). Hence, the model in Table 7-7 is referred to as Model US- $SS_{WD, BIOV}$ herein.

The validation plot for the Model US- $SS_{WD, BIOV}$ is shown in Figure 7-21. Like that shown in Figure 7-16 for the $SS_{WD, BIOV}$ model, Figure 7-21 also shows the large scatter among the data points corresponding to the predicted capacity of approximately 40 kips (178 kN). This scatter in the validation plot is not due to the model inadequacy. This is due to the inherent scatter in $C_{T,SS}$ under identical exposure conditions. Therefore, it can be concluded that the model is homoskedastic. It should also be noted that the 1:1 line passes through the mean value of $C_{T,SS}$ as it approaches $MUTS_{ARS}$. Also, most data points lie within and some lie outside the $\pm\sigma$ region, agreeing to the normality assumption.

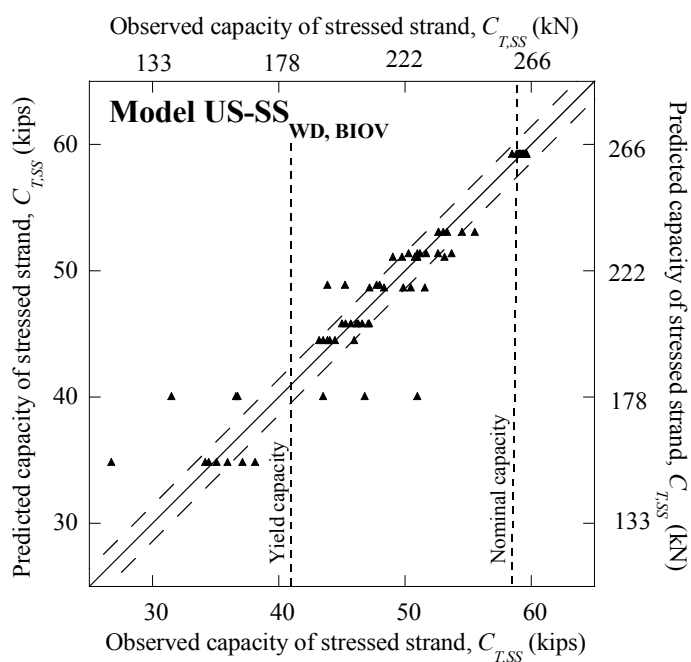


Figure 7-21. Validation Plots for US- $SS_{WD, BIOV}$ Model.

7.5.4.6. Prediction of tension capacity of stressed strands

Figure 7-17(a), (b), (c), and (d) in Subsection 7.4.3 provided $C_{T,SS}$ that were extrapolated or predicted using SS_{WD} models. Figure 7-22(a), (b), (c), and (d) show $C_{T,SS}$ for WD conditions with 0.006, 0.018, 0.18, and 1.8 % sCl^- solutions, respectively. These $C_{T,SS}$ values are extrapolated or predicted using US- SS_{WD} models and are based on the assumption that ϕ_{wet} is equal to 0.17 (see Subsection 7.4.3 for details).

Figure 7-22(a) and (b) show that negligible reduction in $C_{T,SS}$ may be expected when NV and 0.006 or 0.18 % sCl^- conditions exist for approximately 20 years. Figure 7-22(c) and (d) show that, in the case of NV conditions, the exposure to 0.18 and 1.8 % sCl^- solutions (i.e., moderate and high chloride conditions) can cause C_T to drop below $MUTS_{ARS}$ (or nominal capacity) within 7 and 1 year, respectively. From Figure 7-22(a), (b), (c), and (d), it can be concluded that, for PV and BIOV conditions, the predicted $C_{T,SS}$ can, in general, fall below $MUTS_{ARS}$ within approximately 1 year. For PV samples exposed to 0.006 % sCl^- condition, $C_{T,SS}$ dropped below $MUTS_{ARS}$ within approximately 2 years. Figure 7-22(a), (b), (c), and (d) show that when BIOV and WD conditions exist, $C_{T,SS}$ can drop to a value below yield capacity within approximately 9, 7, 5, and 4 years (shown by vertical dashed arrows) when exposed to 0.006, 0.018, 0.18, and 1.8 % sCl^- solutions, respectively.

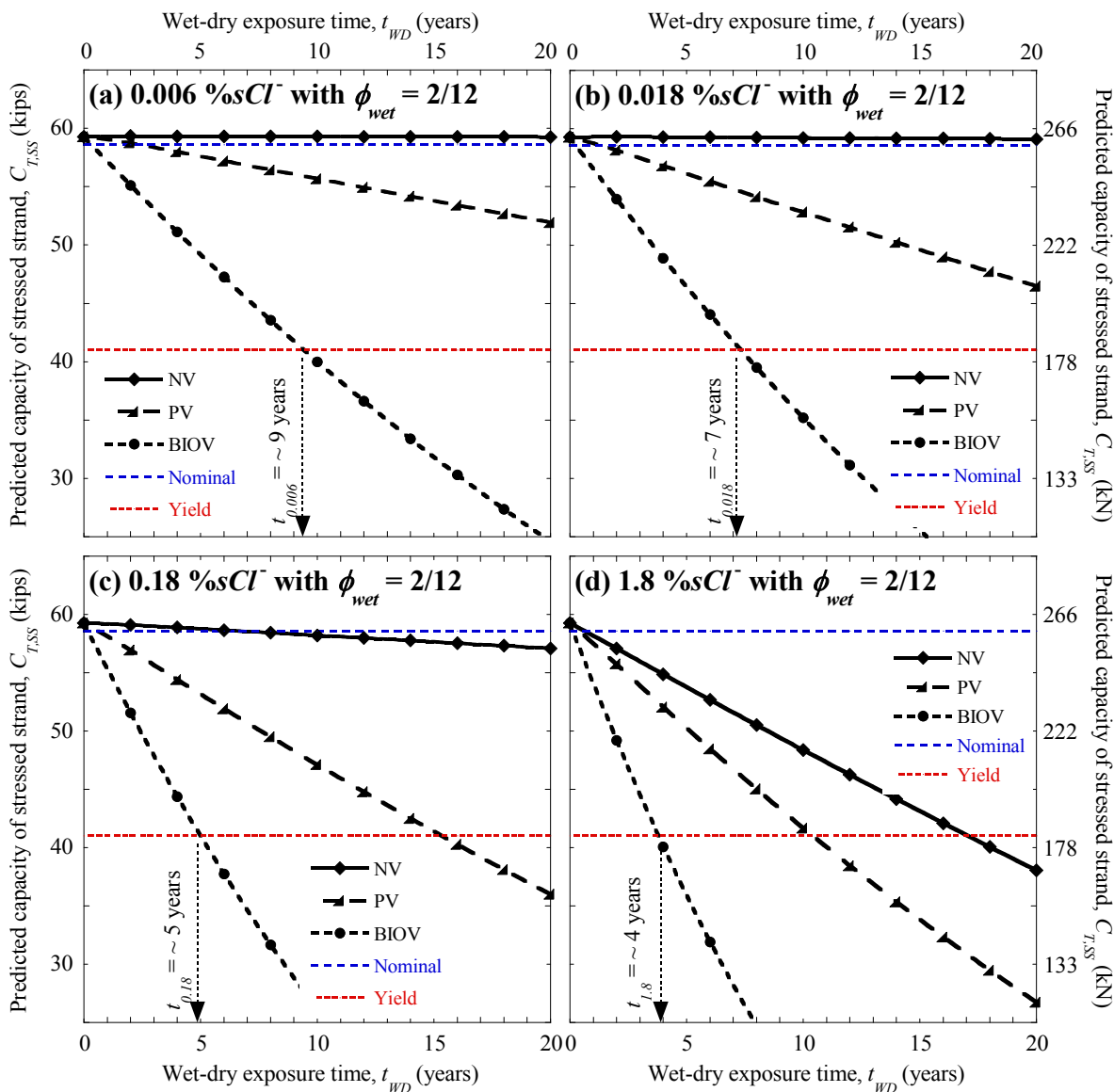


Figure 7-22. Capacity of Stressed Strands under WD Exposure ($\phi_{wet} = 0.17$) to Various Chloride Solutions (Predicted Using US-SSWD Models).

7.5.5. STEP WD-5: Selection of the most appropriate set of models for WD conditions

Now we have three sets of models (i.e., US_{WD,NV}, SS_{WD,NV}, and US-SS_{WD,NV} models) for predicting the capacity of strands under WD exposure conditions. Among these, the best set of models has to be selected. The MAPE and σ of all the models developed for WD exposure conditions are summarized in Table 7-11 and Table 7-12, respectively. Because of the

significant influence of the f_{axial} , the SS_{WD} models are better than the US_{WD} models to predict the C_T of stressed strands in PT bridges. However, a scientific choice between the SS_{WD} and $US-SS_{WD}$ models has to be made. Table 7-11 and Table 7-12 show that the MAPE and σ , respectively, of SS_{WD} models are almost identical with those of the corresponding $US-SS_{WD}$ models.

Table 7-11. MAPEs of US_{WD} , SS_{WD} , and $US-SS_{WD}$ Models.

Exposure condition	MAPE (%) of			Increase in MAPE [$\Delta = (US-SS) - SS$]
	US model	SS model	US-SS model	
WD, NV	1.22	0.73	0.73	0.00
WD, PV	1.61	1.89	1.93	0.04
WD, BIOV	3.15	3.25	3.15	-0.10

Table 7-12. Model Error Coefficients (σ) of US_{WD} , SS_{WD} , and $US-SS_{WD}$ Models.

Exposure condition	Model error coefficient (σ) of			Increase in MAPE [$\Delta = (US-SS) - SS$]
	US model	SS model	US-SS model	
WD, NV	0.0194	0.0121	0.0117	-0.0004
WD, PV	0.0256	0.0249	0.0244	-0.0005
WD, BIOV	0.0350	0.0427	0.0411	-0.0016

Hence, the models may be selected based on the statistical sample size used and the convenience for future research and development. The $US_{WD, NV}$, $US_{WD, PV}$, and $US_{WD, BIOV}$ models were developed based on larger sample sizes (i.e., 105, 100, and 239 unstressed strand samples, respectively) than those used for developing $SS_{WD, NV}$, $SS_{WD, PV}$, and $SS_{WD, BIOV}$ models (i.e., 60, 85, and 79 stressed strand samples, respectively). Although the SS_{WD} models exhibited reasonably good prediction accuracy, a set of three $US-SS_{WD}$ models were also developed based on $C_{T,SS}$ data and the corresponding $C_{T,US}$ predicted using US_{WD} models. Hence, effectively, all the experimental observations are used in developing the $US-SS_{WD}$ models. In the future, the

existing US_{WD} models may be improved by performing less expensive unstressed strand tests with a wider spectrum of environmental conditions. Performing stressed strand tests is very expensive and cumbersome. Fortunately, modified models for stressed strands may be developed by simply replacing the predictor parameter for the US-SS_{WD} models with the modified US_{WD} models (instead of performing expensive stressed strand tests). Based on these reasons, the US-SS_{WD} models are selected as the set of probabilistic models for strands under WD exposure conditions.

7.5.6. Summary of strand capacity models for wet-dry conditions

Probabilistic models for C_T of unstressed and stressed strands that are exposed to WD conditions were developed. A total of nine models (i.e., three US_{WD}, three SS_{WD}, and three US-SS_{WD} models) were developed. These nine models are summarized as follows:

$$\begin{aligned}
 \text{Model US}_{\text{WD,NV}} : R_{C_T,US} &= 1.0105 - 0.0000\gamma_{t_{\text{WD}}} - 1.6785\gamma_{\%sCI^-}\gamma_{t_{\text{WD}}} + 0.0194\varepsilon \\
 \text{Model US}_{\text{WD,PV}} : R_{C_T,US} &= 1.0232 - 0.1553\gamma_{t_{\text{WD}}} - 0.0153 \ln[\gamma_{\%sCI^-}]\gamma_{t_{\text{WD}}} + 0.0256\varepsilon \\
 \text{Model US}_{\text{WD,BIOV}} : R_{C_T,US} &= 1.0333 - 0.3567\gamma_{t_{\text{WD}}} - 0.0285 \ln[\gamma_{\%sCI^-}]\gamma_{t_{\text{WD}}} + 0.0350\varepsilon
 \end{aligned} \tag{7.16}$$

$$\begin{aligned}
 \text{Model SS}_{\text{WD,NV}} : R_{C_T,SS} &= 1.0126 - 0.0000\gamma_{t_{\text{WD}}} - 2.2542\gamma_{\%sCI^-}\gamma_{t_{\text{WD}}} + 0.0121\varepsilon \\
 \text{Model SS}_{\text{WD,PV}} : R_{C_T,SS} &= 1.0129 - 0.2719\gamma_{t_{\text{WD}}} - 0.0277 \ln[\gamma_{\%sCI^-}]\gamma_{t_{\text{WD}}} + 0.0249\varepsilon \\
 \text{Model SS}_{\text{WD,BIOV}} : R_{C_T,SS} &= 1.0095 - 0.6274\gamma_{t_{\text{WD}}} - 0.0485 \ln[\gamma_{\%sCI^-}]\gamma_{t_{\text{WD}}} + 0.0427\varepsilon
 \end{aligned} \tag{7.17}$$

Model US – SS_{WD,NV} :-

$$R_{C_T,SS} = 0.9983 \left[1.0105 - 0.0000\gamma_{t_{\text{WD}}} - 1.6785\gamma_{\%sCI^-}\gamma_{t_{\text{WD}}} \right]^{1.3576} + 0.0117\varepsilon$$

Model US – SS_{WD,PV} :-

$$R_{C_T,SS} = 0.9748 \left[1.0232 - 0.1553\gamma_{t_{\text{WD}}} - 0.0153 \ln[\gamma_{\%sCI^-}]\gamma_{t_{\text{WD}}} \right]^{1.8139} + 0.0256\varepsilon \tag{7.18}$$

Model US – SS_{WD,BIOV} :-

$$R_{C_T,SS} = 0.9463 \left[1.0333 - 0.3567\gamma_{t_{\text{WD}}} - 0.0285 \ln[\gamma_{\%sCI^-}]\gamma_{t_{\text{WD}}} \right]^{2.0301} + 0.0350\varepsilon$$

where the terms are as defined in Equation (7.1) and Subsection 7.4.1. These models were developed based on statistical procedures provided in Subsection 7.2 and the experimental data from strand corrosion tests. Then the developed US_{WD} , SS_{WD} , and $US-SS_{WD}$ models were used to predict C_T of strands under WD exposure conditions. Finally, $US-SS_{WD}$ models were selected as the most appropriate set of models to predict C_T under WD exposure conditions.

8. MODELING THE RELATIONSHIPS BETWEEN TENSION CAPACITIES OF STRANDS AND WIRES

Equation Section (Next)

8.1. INTRODUCTION

Theoretical models relating tension capacity (C_T) of strands and wires exposed to similar conditions could not be found in literature. This section first presents the analytical program to develop probabilistic relationships between the C_T of strands and wires exposed to WD conditions. This analytical program uses the data obtained from the experimental program in discussed in Section 1 and statistical procedures discussed in Subsection 7.2. Following this, two models relating the C_T of stressed strands and unstressed wires are formulated and assessed.

8.2. RELATIONSHIPS BETWEEN TENSION CAPACITIES OF STRESSED STRANDS AND UNSTRESSED WIRES – ANALYTICAL PROGRAM

8.2.1. Introduction and objectives

The objective of this subsection is to present an analytical program to develop a probabilistic model to predict C_T of stressed strand from C_T of unstressed wire. This model is developed for BIOV conditions only. To provide the reader with a sense of expected results, preliminary discussions of possible probabilistic model forms are also provided in this section. The developed probabilistic models are presented in Section 8.3. The parameters used in these probabilistic models are presented next.

8.2.2. Parameters used in the probabilistic models for tension capacity

Subsection 7.2.2 suggested using dimensionless response and predictor parameters to improve the range of application of probabilistic models. Based on this suggestion, the response and predictor parameters, except t_{WD} , are standardized. Because the strand manufacturer did not provide the nominal capacity of king-wires, $MUTS_{ARS}$ is used in modeling C_T of wires (i.e.,

king-wires) instead of nominal tension capacity of “as-received” wires. The following equations define these basic response and predictor parameters, which are used in probabilistic modeling of C_T .

$$R_{C_{T,US}} = \frac{\text{Tension capacity of unstressed strand}}{MUTS_{ARS}} = \frac{C_{T,US}}{MUTS_{ARS}} \quad (8.1)$$

$$R_{C_{T,SS}} = \frac{\text{Tension capacity of stressed strand}}{MUTS_{ARS}} = \frac{C_{T,SS}}{MUTS_{ARS}} \quad (8.2)$$

$$R_{C_{T,UW}} = \frac{\text{Tension capacity of unstressed wire}}{MUTS_{ARS}} = \frac{C_{T,UW}}{MUTS_{ARS}} \quad (8.3)$$

$$\begin{aligned} \gamma_{t_{WD}} &= \left(\frac{\text{Average wet-time in a year (months)}}{12 \text{ (months)}} \right) \times \text{Total wet-dry exposure time (years)} \\ &= \phi_{wet} \times t_{WD} \text{ (years); } 0 \leq \phi_{wet} \leq 1 \end{aligned} \quad (8.4)$$

$$\gamma_{\%sCl^-} = \frac{\%Cl^- \text{ in the exposure solution}}{\%sCl^-_{\text{saturated chloride solution}}} = \frac{\%sCl^-}{35.7} \quad (8.5)$$

Probabilistic models for C_T of strands under WD and CA exposure conditions are developed using suitable transformations (such as exponential, logarithmic, etc.) of the basic response and predictor parameters defined and the statistical procedures provided in Subsection 7.2. The next three subsections discuss the analytical steps involved in this process.

8.2.3. Analytical steps to develop “unstressed wire-stressed strand” capacity model (steps WS-1 through WS-4)

In the previous subsection, the analytical steps to develop probabilistic models for C_T of strands under WD exposure conditions were discussed. In this subsection, the analytical steps to develop probabilistic models to predict $C_{T,SS}$ from $C_{T,UW}$ for BIOV exposure conditions are provided. Table 8-1 shows the list of probabilistic models discussed in this subsection.

Table 8-1. List of Wire and “Wire-Strand” Capacity Models for BIOV Conditions.

Expanded model name	Abbreviated model name
Unstressed Wire _{Wet-Dry, Bleedwater, Inclined, and Orthogonal Void} (for 0.006, 0.018, and 1.8 %sCl ⁻)	UW _{WD, BIOV} (UW _{0.006, BIOV} , UW _{0.018, BIOV} , UW _{1.8, BIOV})
Unstressed Wire-Unstressed Strand _{Wet-Dry, Bleedwater, Inclined, and Orthogonal Void}	UW-US _{WD, BIOV}
Unstressed Wire-Unstressed Strand-Stressed Strand _{Wet-Dry, Bleedwater, Inclined, and Orthogonal Void}	UW-US-SS _{WD, BIOV}
Unstressed Wire-Stressed Strand _{Wet-Dry, Bleedwater, Inclined, and Orthogonal Void}	UW-SS _{WD, BIOV}
Unstressed Wire-Stressed Strand _{Bleedwater, Inclined, and Orthogonal Void}	UW-SS _{BIOV}

Based on the fact that the cross-sectional area of wires and strands are linearly related, their capacities could be linearly related, if exposed to identical conditions. However, C_T of strands might not be directly proportional to C_T of wires because of the flower-like cross-section of strands instead of a solid circular cross-section and the ongoing complex corrosion mechanisms (for example, crevice corrosion due to flower-like cross-section of strands, pitting corrosion due to chlorides, localized corrosion due to GAS interfaces, etc.). Therefore, the data from both strand and wire corrosion tests under WD exposure conditions are used in developing these models. Figure 8-1 shows the flowchart with four steps (i.e., WS-1 through WS-4 [WS indicating “wire-strand”]) for developing the UW-SS_{BIOV} model. In Figure 8-1, the boxes with thick border lines indicate these four steps. The inner boxes with rounded corners provide preliminary information on the results from each step. These four analytical steps are also graphically represented in Figure 8-2 and Figure 8-3(b) and are discussed next.

Step #WS-1

Develop separate tension capacity models (i.e., Model $UW_{WD, BIOV}$) for wires under 0.006, 0.018, and 1.8 % sCl conditions.

$$R_{C_{T,UW}}(\mathbf{x}, \Theta) = \theta_0 + \theta_1 \gamma_{l_{WD}} + \sigma \varepsilon; \text{ for } 0.006, 0.018, \text{ and } 1.8 \% sCl \text{ and } BIOV \text{ conditions}$$

Step #WS-2

Develop $UW-US-SS_{WD, BIOV}$ model by combining $UW-US_{WD, BIOV}$ and $US-SS_{WD, BIOV}$ models.

Step #WS-2a

Develop the $UW-US_{WD, BIOV}$ model by regressing the mean predicted unstressed wire capacity against the observed unstressed strand capacity.

$$R_{C_{T,US}}(\mathbf{x}, \Theta) = \theta_{0,a} (\gamma_{C_{T,UW}})^{\theta_{1,a}} + \sigma_a \varepsilon_a; \text{ for } BIOV \text{ conditions}$$

Step #WS-2b

Develop $UW-US-SS_{WD, BIOV}$ model by nesting the $UW-US_{WD, BIOV}$ model into the $US-SS_{WD, BIOV}$ model.

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) = \theta_0 \left[\theta_{0,a} (\gamma_{C_{T,UW}})^{\theta_{1,a}} \right]^{\theta_1} + \sigma \varepsilon; \text{ for } BIOV \text{ conditions}$$

Step #WS-3

Develop $UW-SS_{WD, BIOV}$ model using the median predicted unstressed wire capacity (from $UW_{WD, BIOV}$ model) and the observed stressed strand capacity.

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) = \theta_0 (\gamma_{C_{T,UW}})^{\theta_1} + \sigma \varepsilon; \text{ for } BIOV \text{ conditions}$$

Step #WS-4

Define the more suitable model from $UW-US-SS_{WD, BIOV}$ and $UW-SS_{WD, BIOV}$ models as $UW-SS_{BIOV}$ model.

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) = \theta_0 (\gamma_{C_{T,UW}})^{\theta_1} + \sigma \varepsilon; \text{ for } BIOV \text{ conditions}$$

Figure 8-1. Flowchart for Developing the “Wire-Strand” Capacity Relationships.

Step WS-1: Develop $UW_{WD, BIOV}$ models for C_T of wires under WD and BIOV conditions (see first box in Figure 8-1 and Figure 8-2). The experimental design was shown in Figure 5-8(c) and Table 5-12. For convenience, Figure 5-8(c) is reproduced in Figure 8-2(a). To

minimize the error in prediction, three individual models for 0.006, 0.018, and 1.8 % sCI^- levels are developed with only t_{WD} as a predictor parameter, instead of a general $UW_{WD, BIOV}$ model with both t_{WD} and % sCI^- as predictor parameters. Each rectangle in Figure 8-2(a) indicates the experimental data used to develop these three individual models that are denoted as Model $UW_{0.006, BIOV}$, $UW_{0.018, BIOV}$, and $UW_{1.8, BIOV}$, respectively. The numbers in the subscripts indicate the % sCI^- level. These models are then used to predict the C_T of wires at 12, 16, and 21 months of exposure, as shown by the hollow circles within the rectangles in Figure 8-2(b). These are the times at which stressed strands were tested.

Step WS-2: Develop the $UW-US-SS_{WD, BIOV}$ model to predict $C_{T,SS}$ using the observed $C_{T,US}$ and $C_{T,SS}$ and the predicted $C_{T,UW}$ (see second box in Figure 8-1 and Figure 8-3[a] and [b]). This model is developed using the following substeps:

Substep WS-2a: Develop the $UW-US_{WD, BIOV}$ model using the predicted $C_{T,UW}$ (from $UW_{WD, BIOV}$ models) and observed $C_{T,US}$. These predicted and observed C_T values are depicted through the two vertical rectangles combining Figure 8-3(a) and (b). Note that Figure 8-3(b) is identical to Figure 8-2(b).

Substep WS-2b: Develop the $UW-US-SS_{WD, BIOV}$ model by nesting the $UW-US_{WD, BIOV}$ model (from substep #WS-2a) into the $US-SS_{WD, BIOV}$ model (see step #WD-4).

Step WS-3: Develop the $UW-SS_{WD, BIOV}$ model using the observed $C_{T,SS}$ and corresponding $C_{T,UW}$ predicted using $UW_{WD, BIOV}$ models. This is depicted through the three ellipses connecting Figure 8-3(b) and (c).

Step WS-4: Select the more suitable model from steps #WS-2 and #WS-3 and define it as $UW-SS_{BIOV}$ model, which can be used for both WD and CA exposure conditions.

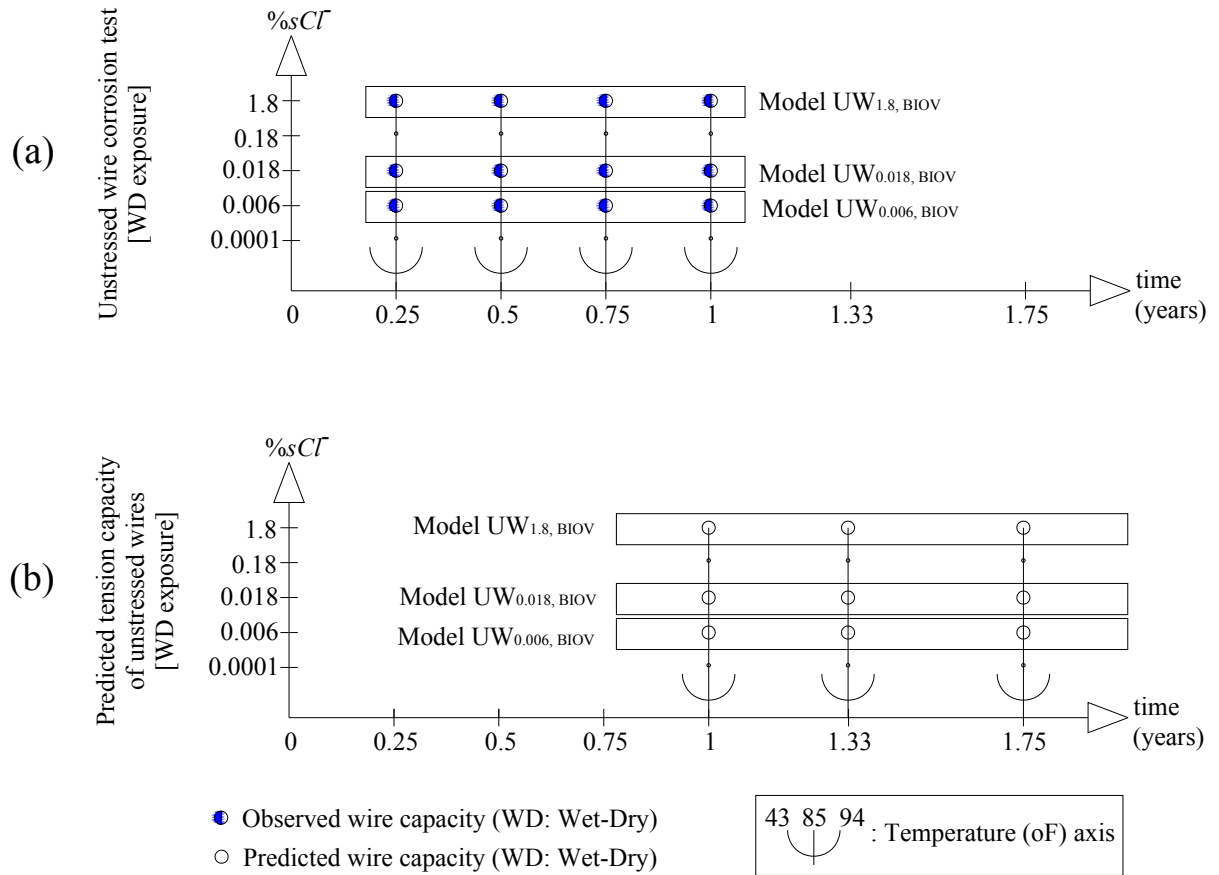


Figure 8-2. Graphical Representation of Step WS-1.

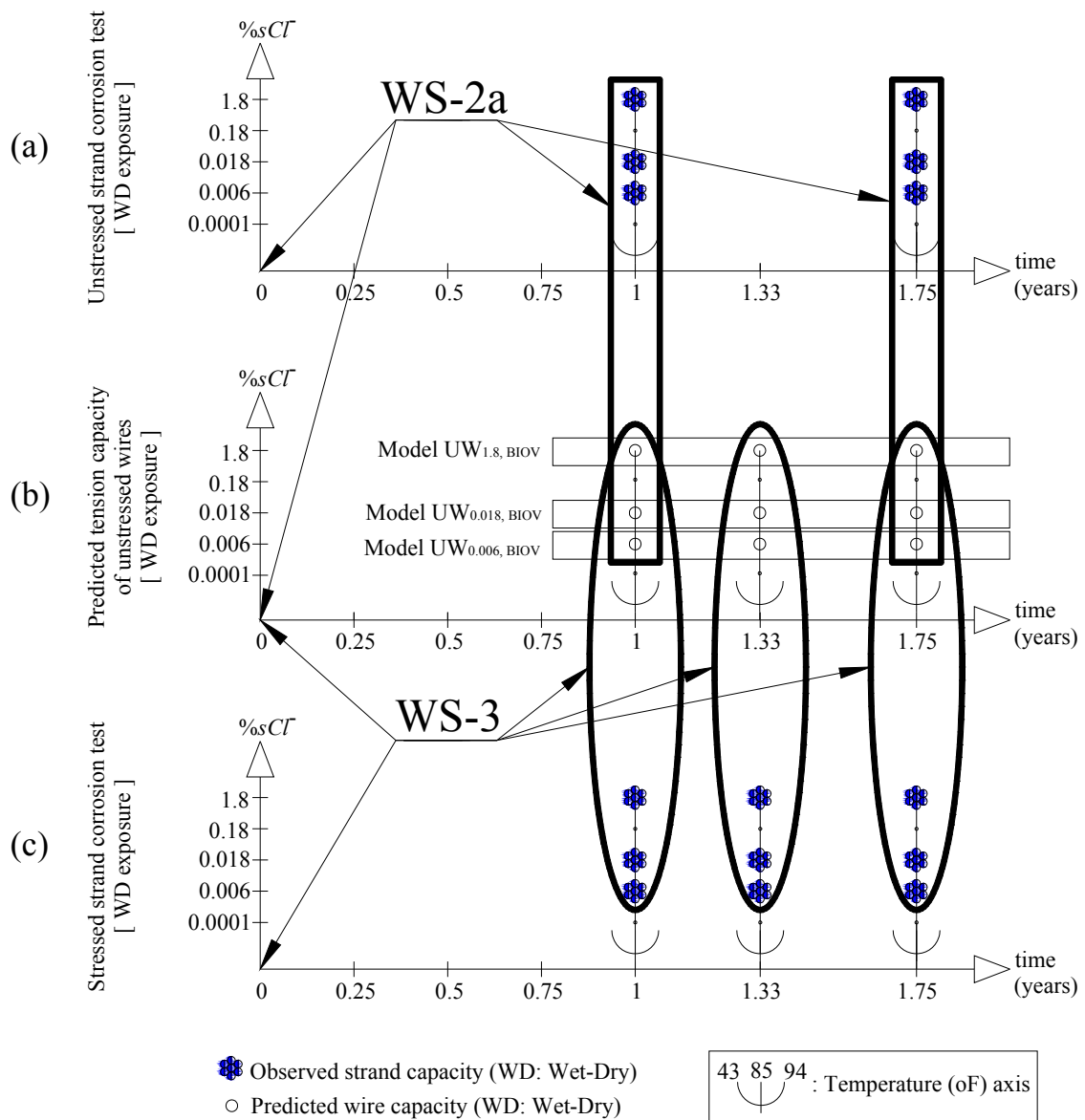


Figure 8-3. Graphical Representations of Steps WS-2a and WS-3.

8.3. RELATIONSHIPS BETWEEN TENSION CAPACITIES OF STRESSED STRANDS AND UNSTRESSED WIRES – RESULTS

The analytical steps (WS-1 through WS-4) to develop probabilistic relationships between C_T of strands and wires under WD exposure conditions were outlined in Subsection 8.2. These analytical steps are executed using the data presented in Subsection 6.4.3 and are presented next.

8.3.1. STEP WS-1: Develop models for wires under 0.006, 0.018, and 1.8 %s Cl^- levels

8.3.1.1. *Diagnosis of the effect of wet-dry exposure time*

As a first step in the modeling process, diagnostic plots are developed to study the effect of $\gamma_{t_{WD}}$ on $C_{T,UW}$. Figure 8-4(a), (b), and (c) clearly show that $C_{T,UW}$ can be modeled as a linear function of $\gamma_{t_{WD}}$ when exposed to 0.006, 0.018, and 1.8 %s Cl^- conditions, respectively. Hence, $\gamma_{t_{WD}}$ is incorporated into the formulated model provided in the next subsection. As shown in Figure 8-4(a), the rate of capacity loss has reduced after 9 months when exposed to 0.006 and 0.018 %s Cl^- , respectively. In Figure 8-4(b), $C_{T,UW}$ at 12 months is larger than that at 9 months. Only three samples were tested for each variable combination, and odd observations like this (in Figure 8-4[b]) might be misleading and need to be avoided, especially when statistical sample size is small and no scientific explanations are available. However, as shown in Figure 8-4(c), when the %s Cl^- was very high (i.e., 1.8 %s Cl^-), the rate of capacity loss did not change much even after 9 months of exposure.

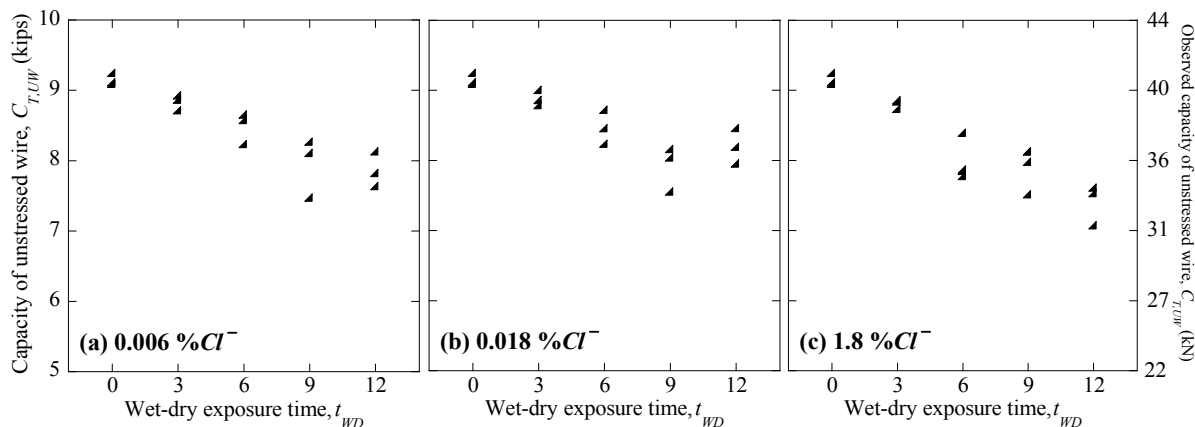


Figure 8-4. Scatter Plot between Capacity of Unstressed Wires, $C_{T,UW}$, and Wet-Dry Exposure Time, t_{WD} .

8.3.1.2. Formulation and assessment

Following the general model form (Equation (7.1)), the probabilistic models for $C_{T,UW}$ under WD exposure conditions can be expressed as follows:

$$R_{C_{T,UW}}(\mathbf{x}, \Theta) = \theta_0 + \theta_1 \gamma_{t_{WD}} + \sigma \varepsilon \quad (8.6)$$

where the terms are as defined in Equation (7.1) and Subsection 7.4.1. The unknown model parameters in Equation (8.6) were assessed using the statistical procedures provided in Subsection 7.2. Table 8-2 summarizes the MAPE and posterior statistics of these three linear models (i.e., for 0.006, 0.018, and 1.8 % sCl^- conditions) with only $\gamma_{t_{WD}}$ as the predictor variable. The MAPE values of $UW_{WD,0.006}$, $UW_{WD,0.018}$, and $UW_{WD,1.8}$ models are 1.92, 2.19, and 2.77 percent, respectively, indicating reasonably good prediction for practical engineering purposes. Also, the MAPE values increase as the % sCl^- increases. This is because of the inherent increase in the scatter induced by the presence of increasing amount of chlorides on the steel surface. As in Table 8-2, the most unfavorable COV among θ_i is -0.20 (i.e., for θ_1 in $UW_{WD,0.018}$ model). This is due to the unexpected fact, as shown in Figure 8-4(b), that the observed capacities at 12 months were larger than those at 9 months. However, the COV of σ

for these three models are 0.68, 0.69, 0.66, respectively. These values are very high because of the inherent scatter among the observed capacities exhibited by the samples' exposed identical conditions and the small statistical sample size. However, the small MAPE values suggest that the model can provide sufficiently accurate prediction for practical engineering purposes.

Table 8-2. MAPE and Posterior Statistics of UW_{WD} Models.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation	
						θ_0	θ_1
$UW_{WD, 0.006}$	1.93	θ	0.1562	0.0020	0.01	1	
		θ_1	-0.0018	0.0003	-0.17	-0.82	1
		σ	0.0044	0.0030	0.68		
$UW_{WD, 0.018}$	2.19	θ	0.1553	0.0023	0.01	1	
		θ_1	-0.0015	0.0003	-0.20	-0.82	1
		σ	0.0052	0.0036	0.69		
$UW_{WD, 1.8}$	2.77	θ	0.1555	0.0022	0.01	1	
		θ_1	-0.0023	0.0003	-0.13	-0.84	1
		σ	0.0050	0.0033	0.66		

Figure 8-5, Figure 8-6, and Figure 8-7 show the validation plots for $UW_{WD, 0.006}$, $UW_{WD, 0.018}$, and $UW_{WD, 1.8}$ models. The details on analyzing validation plots are provided in Subsection 7.2.3.5. In general, the data in Figure 8-5, Figure 8-6, and Figure 8-7 are spread in an approximately uniform width along the 1:1 line and show no signs of residual trend or bias in the overall prediction. In Figure 8-5 and Figure 8-6, slight deviations are observed for points corresponding to $C_{T,UW}$ values less than or equal to 8 kips (36 kN). However, these can be attributed to the unexpected deviations observed in the diagnostic plots in Figure 8-4(a) and (b). Following the procedures in Subsection 7.2.3.5, these indicate that homoskedasticity assumptions are approximately satisfied. Also, some data points are lying outside the $\pm\sigma$ region, indicating approximately normal distribution of data.

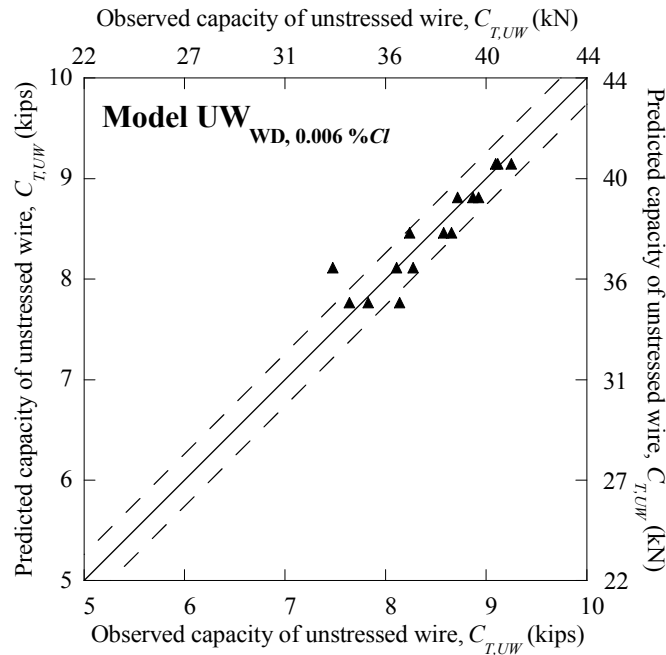


Figure 8-5. Validation Plot for the UW_{WD, 0.006} Model.

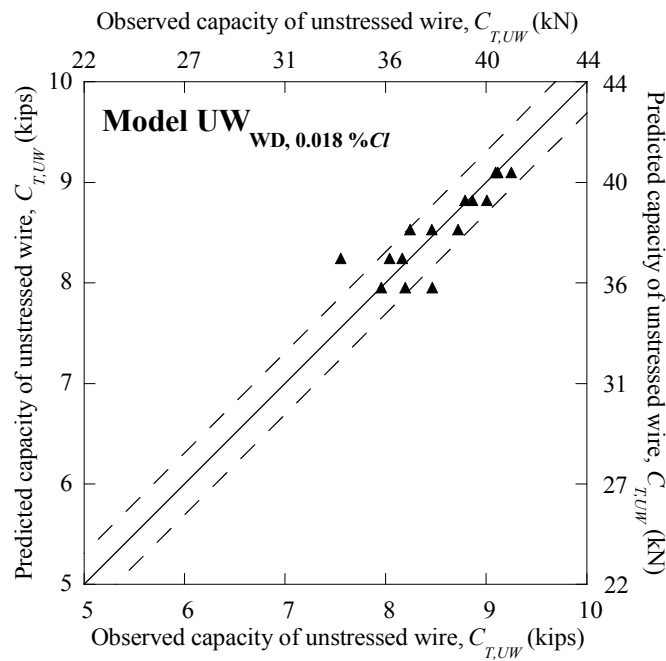


Figure 8-6. Validation Plot for the UW_{WD, 0.018} Model.

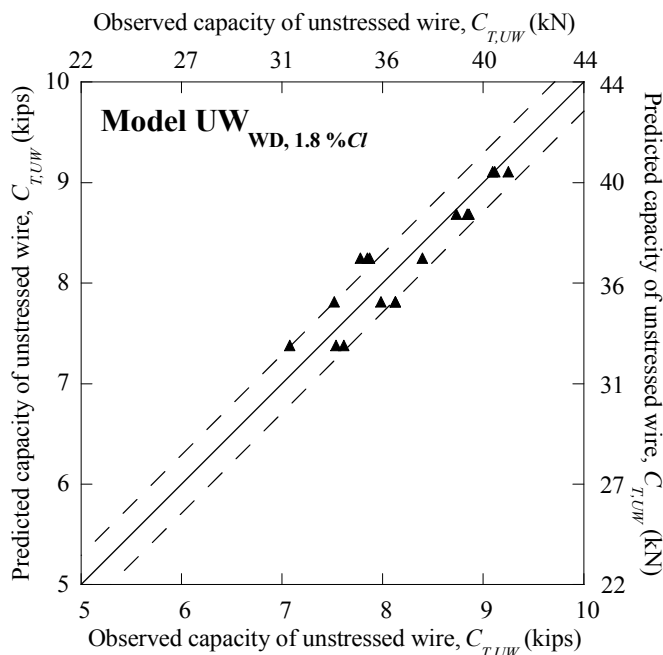


Figure 8-7. Validation Plot for the $UW_{WD, 1.8}$ Model.

The $UW_{WD, 0.006}$, $UW_{WD, 0.018}$, and $UW_{WD, 1.8}$ models can be used to predict $C_{T,UW}$ at 12, 16, and 21 months of WD exposure and can then be compared with $C_{T,US}$ and $C_{T,SS}$ at 12, 16, and 21 months of WD exposure to develop the $UW-US-SS_{WD, BIOV}$ and $UW-SS_{WD, BIOV}$ models in the following subsections.

8.3.2. STEP WS-2: Develop $UW-US-SS_{WD, BIOV}$ model

As shown in Figure 8-1, the $UW-US-SS_{WD}$ model is developed in two substeps as discussed next.

8.3.2.1. Step WS-2a: Develop Model $UW-US_{WD, BIOV}$

Figure 8-8 shows the diagnostic plot between $C_{T,US}$ and $C_{T,UW}$. $C_{T,UW}$ plotted in Figure 8-8 is predicted using UW_{WD} models (Equation (8.6)). Figure 8-8 shows that a power function of $C_{T,UW}$ is a good approximation and is incorporated into the probabilistic model form provided next.

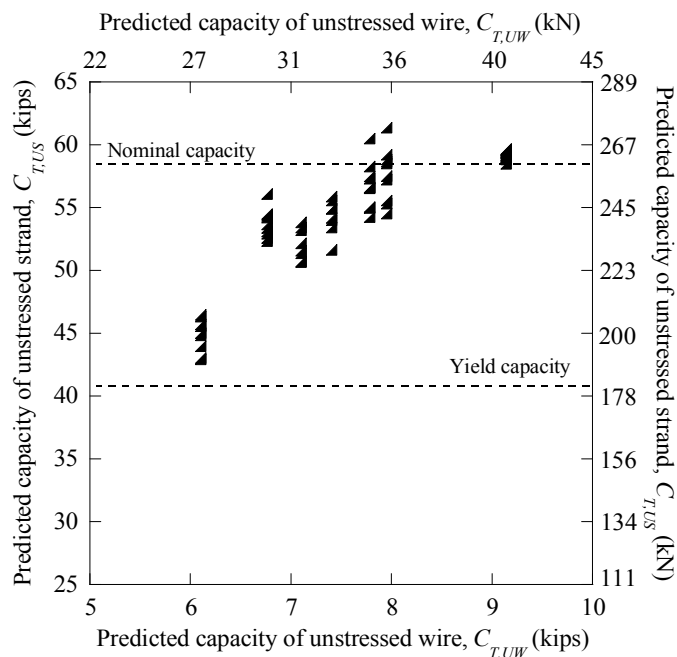


Figure 8-8. Scatter Plot between Predicted Capacity of Unstressed Wires, $C_{T,UW}$, and Observed Capacity of Unstressed Strands, $C_{T,US}$.

The following model form was developed to predict $C_{T,US}$ from $C_{T,UW}$:

$$\text{Model UW-US}_{\text{WD}} : R_{C_{T,US}} = \theta_{0,a} \left(\gamma_{C_{T,UW}} \right)^{\theta_{1,a}} + \sigma_a \varepsilon_a \quad (8.7)$$

where the terms are as defined in Equation (7.1) and Subsection 7.4.1. In order to improve clarity in Step WS-2-c, the subscript “a” has been added to all the model parameters in Equation (8.7). Table 8-3 shows the posterior statistics of the UW-US_{WD, BIOV} model. The MAPE of the full model form is 3.29 percent, which is a reasonable value. In addition, the COVs of all the parameters’ estimates are reasonably small. The model error, σ , is also reasonable (i.e., 0.0374 with a standard deviation of 0.30), considering the amount of data available and the inherent scatter observed among the data points with identical exposure conditions. Because of these reasons, the model is referred to as UW-US_{WD, BIOV} model herein.

Table 8-3. MAPE and Posterior Statistics of UW-US_{WD, BIOV} Full Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i	
						θ_0	θ_1
Model UW-US _{WD, BIOV}	3.29	$\theta_{0,a}$	2.8020	0.1904	0.07	1	
		$\theta_{1,a}$	0.5399	0.0337	0.06	-0.99	1
		σ_a	0.0374	0.0030	0.08		

Figure 8-9 shows the validation plot for the reduced UW-US_{WD, BIOV} model. The details on analyzing validation plots are provided in Subsection 7.2.3.5. The plot indicates that, in general, the Model UW-US_{WD} provides reasonably good estimation of $C_{T,US}$. However, in Figure 8-9, as corrosion is severe (for example, when $C_{T,US}$ is approximately 47 kips [209 kN]), there is a slight leftward deviation from the 1:1 line. In some other instances (for example, at $C_{T,US}$ equal to approximately 50 and 57 kips [222 and 254 kN]) the deviation is slightly rightward. However, these deviations do not seem to be more serious than the inherent scatter among the observed data. Hence, it is concluded that the model approximately satisfies the homoskedasticity assumption. Although not perfect, the nature of data points falling both inside and outside the $\pm\sigma$ region exhibits an approximate agreement with the normality assumption.

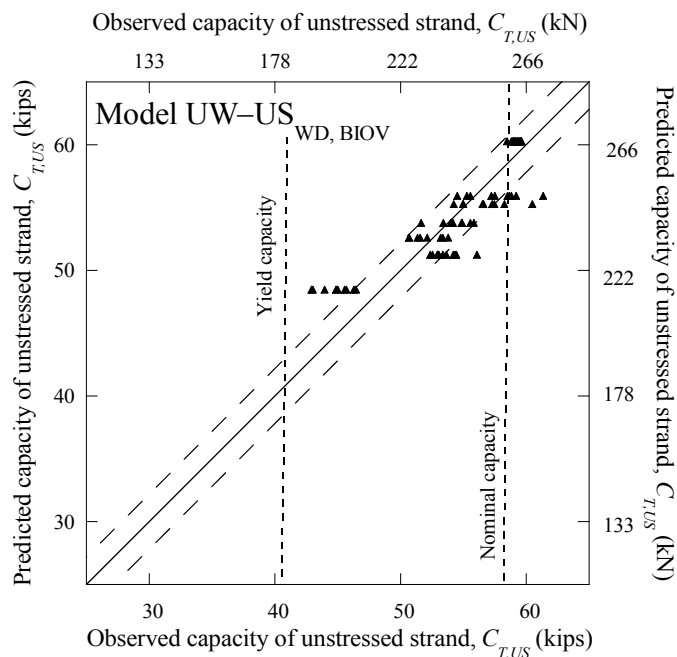


Figure 8-9. Validation Plots for UW-US_{WD} Model.

A model to predict $C_{T,US}$ from $C_{T,UW}$ is now developed. However, in order to use this model to predict $C_{T,SS}$, a model relating the $C_{T,US}$ and $C_{T,SS}$ is needed and is developed next.

8.3.2.2. Step WS-2b: Develop Model UW-US-SS_{WD, BIOV}

The UW-US-SS_{WD, BIOV} model has been developed by nesting the UW-US_{WD, BIOV} model (developed in step WS-2a) into the US-SS_{WD, BIOV} model (developed in step WD-4 in Subsection 7.4.1). For clarity, the US-SS_{WD, BIOV} model is modified with a subscript “b” as follows:

$$R_{C_{T,SS}} = \theta_{0,b} \left(\gamma_{C_{T,US}} \right)^{\theta_{1,b}} + \sigma_b \varepsilon_b \quad (8.8)$$

By substituting the term $\gamma_{C_{T,US}}$ in Equation (8.8) with the UW-US_{WD, BIOV} we get:

$$R_{C_{T,SS}} = \theta_{1,b} \left[\underbrace{\theta_{0,a} \left(\gamma_{C_{T,UW}} \right)^{\theta_{1,a}}}_{\text{median function of Model UW-US}_{WD, BIOV}} \right]^{\theta_{2,b}} + \sigma_b \varepsilon_b \quad (8.9)$$

The error term, $\sigma_a \varepsilon_a$, was removed from the UW-US_{WD, BIOV} model to avoid the issues associated with correlated random variables. By substituting the mean estimates in Table 8-3, the Equation (8.9) becomes:

$$R_{C_{T,SS}} = 0.9463 \left[\underbrace{2.8020 \left(\gamma_{C_{T,UW}} \right)^{0.5399}}_{\text{median function of Model UW-US}_{WD, BIOV}} \right]^{2.0301} + 0.0411 \varepsilon_b \quad (8.10)$$

Equation (8.10) is referred to as Model UW-US-SS_{WD, BIOV} herein. The MAPE of this model is determined to be 7.19 percent. Figure 8-10 is the validation plot for the UW-US-SS_{WD, BIOV} model. The details on analyzing validation plots are provided in Subsection 7.2.3.5. The data points near $MUTS_{ARS}$ are very close to the 1:1 line. However, there is a leftward deviation when capacity is below the yield capacity and a rightward deviation when the capacity is between $MUTS_{ARS}$ and yield strength. Although the $\pm\sigma$ region is drawn, it should be noted that this is an approximation after removing $\sigma_a \varepsilon_a$.

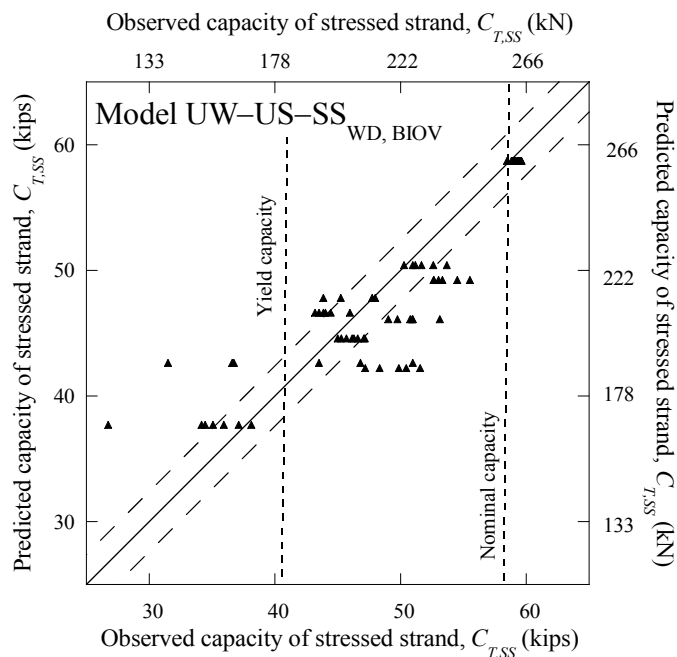


Figure 8-10. Validation Plot for UW-US-SS_{WD, BIOV} Model.

8.3.3. STEP WS-3: Develop UW-SS_{WD, BIOV} Model

This subsection presents the statistical diagnosis of possible predictor parameters for the UW-SS_{WD, BIOV} model. The UW-SS_{WD, BIOV} model is developed directly (i.e., without any intermediate models or substeps as in UW-US-SS_{WD, BIOV} model development) using the observed capacities of unstressed wires, stressed strands, and %sCl⁻ concentrations.

Figure 8-11 shows the scatter plot between $C_{T,SS}$ and the $C_{T,UW}$. Each $C_{T,UW}$ value was predicted using the appropriate UW_{WD} model based on the %sCl⁻ concentrations of each observed value of $C_{T,SS}$. Figure 8-11 indicates that $C_{T,SS}$ can be expressed as a power function of $C_{T,UW}$. This power model form will help the fitted model to pass closely through the $MUTS_{ARS}$.

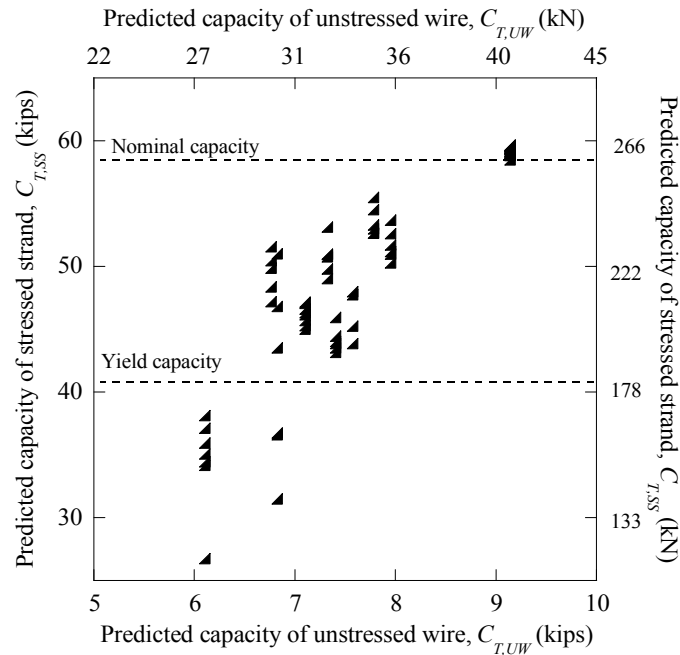


Figure 8-11. Scatter Plot between Predicted Capacity of Unstressed Wires, $C_{T,UW}$, and Observed Capacity of Stressed Strands, $C_{T,SS}$.

Following the general model form (Equation (7.1)) and using the findings in the diagnostic study, the probabilistic models for $C_{T,SS}$ are formulated as follows:

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) = \theta_0 \left(\gamma_{C_{T,UW}} \right)^{\theta_1} + \sigma \varepsilon \quad (8.11)$$

where the terms are as defined in Equation (7.1) and Subsection 7.2. This power model form is similar to that of UW-US-SS_{WD, BIOV} model form (i.e., Equation (8.10)) and supports the fact that $C_{T,SS}$ should be 0 when $C_{T,UW}$ reaches 0. Table 8-4 summarizes the MAPE and posterior statistics of the UW-SS_{WD, BIOV} model, assessed using the procedures provided in Subsection 6.5.

Table 8-4. MAPE and Posterior Statistics of Full UW-SS_{WD, BIOV} Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i	
						θ_0	θ_1
UW-SS _{WD, BIOV}	6.73	θ_0	7.7492	0.9532	0.12	1	
		θ_1	1.0924	0.0617	0.06	0.99	1
		σ	0.0619	0.0047	0.08		

The small COVs, as shown in Table 8-4, indicate that all the model parameters have been estimated with good confidence. Also, the MAPE and σ are reasonably small, considering the scatter in the observed data from the samples with identical exposure conditions. Moreover, the σ of model is 0.0619, a reasonably small value. Because of these reasons, this model is referred to as UW-SS_{WD} herein.

Figure 8-12 is the validation plot showing a comparison between predicted (using Model UW-SS_{WD}) and observed values of $C_{T,SS}$ at t_{WD} equal to 0, 12, 16, and 21 months. The details on analyzing validation plots are provided in Subsection 7.2.3.5. Approximately 85 percent of the data points fall along the 1:1 line and within the $\pm\sigma$ region. This shows a reasonably good model prediction. Also, the data points corresponding to $MUTS_{ARS}$ or nominal capacity (i.e., 58.6 kips [258 kN]) lie very close to the 1:1 line, indicating good prediction in this region. However, as with the UW-US-SS_{WD, BIOV} model, a slight leftward deviation exists near the yield capacity region. As an improvement from the UW-US-SS_{WD, BIOV} model, the UW-SS_{WD, BIOV} model exhibits less rightward shift between the nominal and yield capacity values. Considering the inherent scatter in the corrosion phenomenon, the model seems to exhibit no “serious” systematic bias or residual trend, except when $C_{T,SS}$ is below yield strength. Also, only approximately 17 percent of the data points fall outside the $\pm\sigma$ region, indicating a reasonably good agreement with the normality assumption.

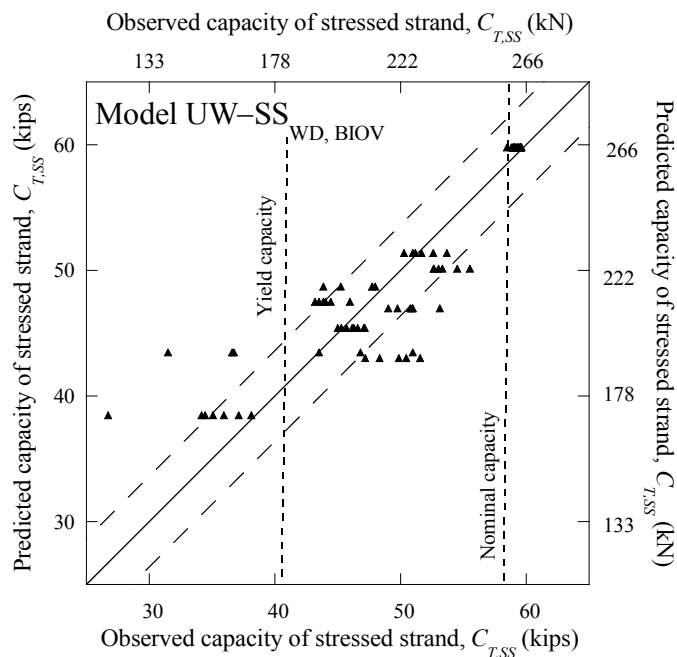


Figure 8-12. Validation Plot for the UW-SS_{WD, BIOV} Model.

8.3.4. Step WS-4: Select the more suitable “wire-strand” relationship for both WD and CA exposure conditions

When comparing Figure 8-10 and Figure 8-12, it is observed that both the UW-US-SS_{WD, BIOV} and UW-SS_{WD, BIOV} models show similar predictions. The MAPE of the UW-SS_{WD, BIOV} model (i.e., 6.73 percent) is slightly less than that of the UW-US-SS_{WD} model (i.e., 7.19 percent). Moreover, the UW-SS_{WD, BIOV} model provides better prediction near the $MUTS_{ARS}$ region than that predicted by the UW-US-SS_{WD, BIOV} model. In addition, the UW-SS_{WD, BIOV} model was developed in a single step, whereas the UW-US-SS_{WD, BIOV} model was developed in multiple steps and involves approximations such as removal of the inner error terms (i.e., $\sigma_a \varepsilon_a$). Because of these reasons, the UW-SS_{WD, BIOV} model is better than the UW-US-SS_{WD, BIOV} model. Because this model can be used for both WD and CA exposure conditions, it will be denoted as UW-SS_{BIOV} model herein and is used in Step CA-3 presented later.

8.3.5. Summary of wire-strand relationships

In Subsection 8.3.1, the $UW_{WD,0.006}$, $UW_{WD,0.018}$, and $UW_{WD,1.8}$ models were developed to predict $C_{T,UW}$. Then using the predicted $C_{T,UW}$ and the observed values of $C_{T,US}$ and $C_{T,SS}$, the $UW-US-SS_{WD,BIOV}$ and $UW-SS_{WD,BIOV}$ models were developed in Subsections 8.3.2 and 8.3.3, respectively, and are summarized as follows:

$$\begin{aligned}
 \mathbf{UW-US-SS}_{WD,BIOV} : R_{C_{T,SS}}(\mathbf{x}, \Theta) &= 0.9463 \underbrace{\left[2.8020 \left(\gamma_{C_{T,UW}} \right)^{0.5399} \right]}_{\text{median function of Model UW-US}_{WD,BIOV}}^{2.0301} + 0.0411 \varepsilon_b \\
 \mathbf{UW-SS}_{WD,BIOV} : R_{C_{T,SS}} &= 7.7492 \left(\gamma_{C_{T,UW}} \right)^{1.0924} + 0.0619 \varepsilon
 \end{aligned} \tag{8.12}$$

Then in Subsection 8.3.4, the $UW-SS_{BIOV}$ model was defined in order to be used in the next subsection to develop the model for $C_{T,SS}$ under CA and BIOV conditions.

9. MODELING TENSION CAPACITY OF STRANDS EXPOSED TO CONTINUOUS-ATMOSPHERIC (CA) CONDITIONS

Equation Section (Next)

9.1. INTRODUCTION

This subsection develops probabilistic models for tension capacity (C_T) of strands under CA exposure conditions. Theoretical models for C_T of strands under CA exposure conditions could not be developed or found in typical literature. Historically, engineers have developed deterministic empirical models when theoretical modeling is not possible. Based on this convention, probabilistic empirical models for C_T of strands under CA conditions are developed using the data obtained from the experimental program and statistical procedures discussed in Subsection 7.2.

9.2. PROBABILISTIC MODELS FOR TENSION CAPACITY OF STRANDS – ANALYTICAL PROGRAM

9.2.1. Introduction and objectives

In this case of CA exposure conditions, it is assumed that an NV condition does not have significant influence on C_T . As observed later in Subsection 6.5.5, a PV condition has less severe effects on C_T than a BIOV condition. Hence, no strand and wire samples were tested under PV and CA exposure conditions and no models were developed for these conditions.

BIOV samples experienced most severe corrosion and capacity loss under WD conditions. However, no strand specimens were tested under BIOV and CA exposure conditions. Because of this, an indirect analytical approach was designed to develop probabilistic models for C_T of strands under BIOV and CA exposure conditions. This indirect approach included two steps as follows: 1) develop a model (denoted as UW-SS_{BIOV} model) to predict C_T of stressed strands from C_T of unstressed wires, and 2) develop a C_T model for unstressed wires exposed to CA conditions and nest it into the UW-SS_{BIOV} model.

The main objective of this subsection is to present analytical steps to develop probabilistic tension capacity models for strands under BIOV and CA exposure conditions. See Subsection 9.2.3 for details. To provide the reader with a sense of expected results, preliminary discussions of possible probabilistic model forms are also provided in this subsection. The developed probabilistic models are presented in Section 9.3. The parameters used in these probabilistic models are presented next.

9.2.2. Parameters used in the probabilistic models for tension capacity

Subsection 7.2.2 suggested using dimensionless response and predictor parameters to improve the range of application of probabilistic models. Based on this suggestion, the response and predictor parameters, except t_{CA} and T , are standardized. The following equations define these basic response and predictor parameters, which are used in probabilistic modeling of C_T .

$$R_{C_T,SS} = \frac{\text{Tension capacity of stressed strand}}{MUTS_{ARS}} = \frac{C_{T,SS}}{MUTS_{ARS}} \quad (9.1)$$

$$R_{C_T,UW} = \frac{\text{Tension capacity of unstressed wire}}{MUTS_{ARS}} = \frac{C_{T,UW}}{MUTS_{ARS}} \quad (9.2)$$

$$\gamma_{t_{CA}} = \text{Total continuous-atmospheric exposure time (years)} \quad (9.3)$$

$$\gamma_T = \text{Ambient exposure temperature (}^\circ\text{F)} = T (^\circ\text{F)} \quad (9.4)$$

$$\gamma_{RH} = \frac{RH (\%)}{\text{Maximum relative humidity (\%)}} = \frac{RH (\%)}{100\%} \quad (9.5)$$

$$\gamma_{\%sCl^-} = \frac{\%Cl^- \text{ in the exposure solution}}{\%sCl^-_{\text{saturated chloride solution}}} = \frac{\%sCl^-}{35.7} \quad (9.6)$$

$$\gamma_{\%gCl^-} = \frac{\%Cl^- \text{ in the grout at or near GAS interface}}{\%sCl^-_{\text{saturated chloride solution}}} = \frac{\%gCl^-}{35.7} \quad (9.7)$$

Using suitable transformations (such as exponential, logarithmic, etc.) of the basic response and predictor parameters defined and the procedures provided in Subsection 7.2, probabilistic models for C_T of strands under WD and CA exposure conditions are developed. The next three subsections discuss the analytical steps involved in this process.

9.2.3. Steps to develop tension capacity models for strands under CA conditions (steps CA-1 through CA-4)

In this subsection, analytical steps to develop C_T of strands under CA exposure conditions will be presented. Most CA exposure conditions found in the field are typically not as aggressive as WD exposure conditions. For example, in the case of strands under NV and CA conditions, corrosion rate is assumed to be negligible. Hence, $C_{T,SS}$ when NV and CA conditions exist can be assumed to be equal to $C_{T,ARS}$. This is shown in step CA-1 discussed later in this subsection.

BIOV and CA exposure conditions might result in aggressive corrosion. As discussed in Subsection 6.5.5, the BIOV condition causes statistically dissimilar and more severe loss in C_T than the PV condition. Moreover, Table 2-1 showed that the probabilities of finding a tendon with PV and BIOV conditions are approximately 6 and 78 percent, respectively. These numbers indicate a much higher importance or need for assessing corrosion of strands with BIOV conditions. In addition, it is highly unlikely that there will be a PV and not a BIOV condition inside a tendon. Based on these field scenarios and experimental convenience, it is concluded that BIOV is the most critical VT influencing C_T of strands exposed to CA conditions. Hence, a probabilistic model for C_T of strands under BIOV and CA conditions only is developed and one for C_T of strands under PV and CA conditions is not developed.

A direct approach of developing probabilistic models for $C_{T,SS}$ under BIOV and CA conditions would be to conduct a long-term stressed strand corrosion test under BIOV and CA conditions. However, under most BIOV and CA exposure conditions, the strands will possibly have to be exposed for several decades in order to cause a capacity reduction that could be reliably determined using conventional tension testing equipment in a laboratory. Hence, this direct approach is time consuming and expensive and, therefore, relatively impractical. An indirect approach has been designed to develop probabilistic models for $C_{T,SS}$ when CA and BIOV conditions exist. This approach uses the $UW-SS_{BIOV}$ model developed in step WS-4, data from the wire corrosion tests performed under CA exposure conditions, and extrapolation procedures. Three steps (i.e., steps CA-2, CA-3, and CA-4) are defined in this approach. Following is a discussion of steps CA-1 through CA-4.

Step CA-1: Develop the $C_{T,CA,NV}$ model to predict $C_{T,SS}$ when CA and NV conditions exist. This model is a logarithmic distribution based on the mean and standard deviation of C_T of “as-received” strands obtained in Subsection 5.2.1.

Step CA-2: Develop the $UW_{CA,BIOV}$ model to predict $C_{T,UW}$ at 9 months of CA exposure (see first box in Figure 9-1 and Figure 9-2[a]). The $UW_{CA,BIOV}$ model is developed using the data from unstressed wire corrosion tests under CA exposure conditions. The experimental design for this was shown earlier in Figure 5-8(d) and Table 5-13. For convenience, Figure 5-8(d) is reproduced in Figure 9-2(a).

Step CA-3: Nest the $UW_{CA,BIOV}$ model (developed in step CA-2) into the $UW-SS_{BIOV}$ model (developed in step WS-4). Use this nested model to predict $C_{T,SS}$ at 9 months of CA exposure (see second box in Figure 9-1 and the strand marker in Figure 9-2[b]). The wire samples were tested only at t_{CA} equal to 9 months and, hence, the $UW_{CA,BIOV}$ model does not contain the time parameter, t_{CA} . To be used in the time-variant reliability models, this predicted $C_{T,SS}$ needs to be extrapolated for future times.

Step CA-4: Develop the $C_{T,CA,BIOV}$ model to predict $C_{T,SS}$ when CA and BIOV conditions exist. This is done by modifying the formulation developed in step CA-3 into a power corrosion model form discussed in Subsections 2.6.2 and 2.6.3 (see third box in Figure 9-1 and the solid curve in Figure 9-2[b]).

Results from steps CA-1 through CA-4 are provided in Subsection 9.3.

Step #CA-1

Develop $C_{T,CA,NV}$ model by expressing the tension capacity of “as-received” strands as a logarithmic distribution.

$$C_{T,SS} = \text{LN}\left(E[C_{T,ARS}], \sqrt{\text{Var}[C_{T,ARS}]}\right); \text{ for NV conditions}$$

Step #CA-2

Develop $UW_{CA,BIOV}$ model to predict tension capacity of unstressed wires at 9 months.

$$R_{C_{T,UW}}(\mathbf{x}, \Theta) \Big|_{t_{CA}=9\text{months}} = \theta_0 + \theta_1 \exp[\gamma_{RH}] + \theta_2 \exp[\gamma_{\%gCI} \exp[\gamma_{RH}] \gamma_T] + \sigma \varepsilon; \text{ for BIOV conditions}$$

Step #CA-3

Predict the tension capacity of stressed strands at 9 months by nesting the $UW_{CA,BIOV}$ model into the $UW-SS_{BIOV}$ model.

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) \Big|_{t_{CA}=9\text{months}} = \theta_0 \left(\gamma_{C_{T,UW}} \Big|_{t_{CA}=9\text{months}} \right)^{\theta_1} + \sigma \varepsilon; \text{ for BIOV conditions}$$

Step #CA-4

Develop $C_{T,CA,BIOV}$ model by modifying the formulation developed in Step CA-2 into a power corrosion model form.

$$R_{C_{T,SS}}(\mathbf{x}, \Theta) = \left[\theta_0 \left(\gamma_{C_{T,UW}} \Big|_{t_{CA}=9\text{months}} \right)^{\theta_1} \right] (t_{CA})^{n_{CA}} + \sigma \varepsilon; \text{ for BIOV conditions}$$

Figure 9-1. Flowchart for Developing the Tension Capacity Models for Strands under CA and BIOV Exposure Conditions.

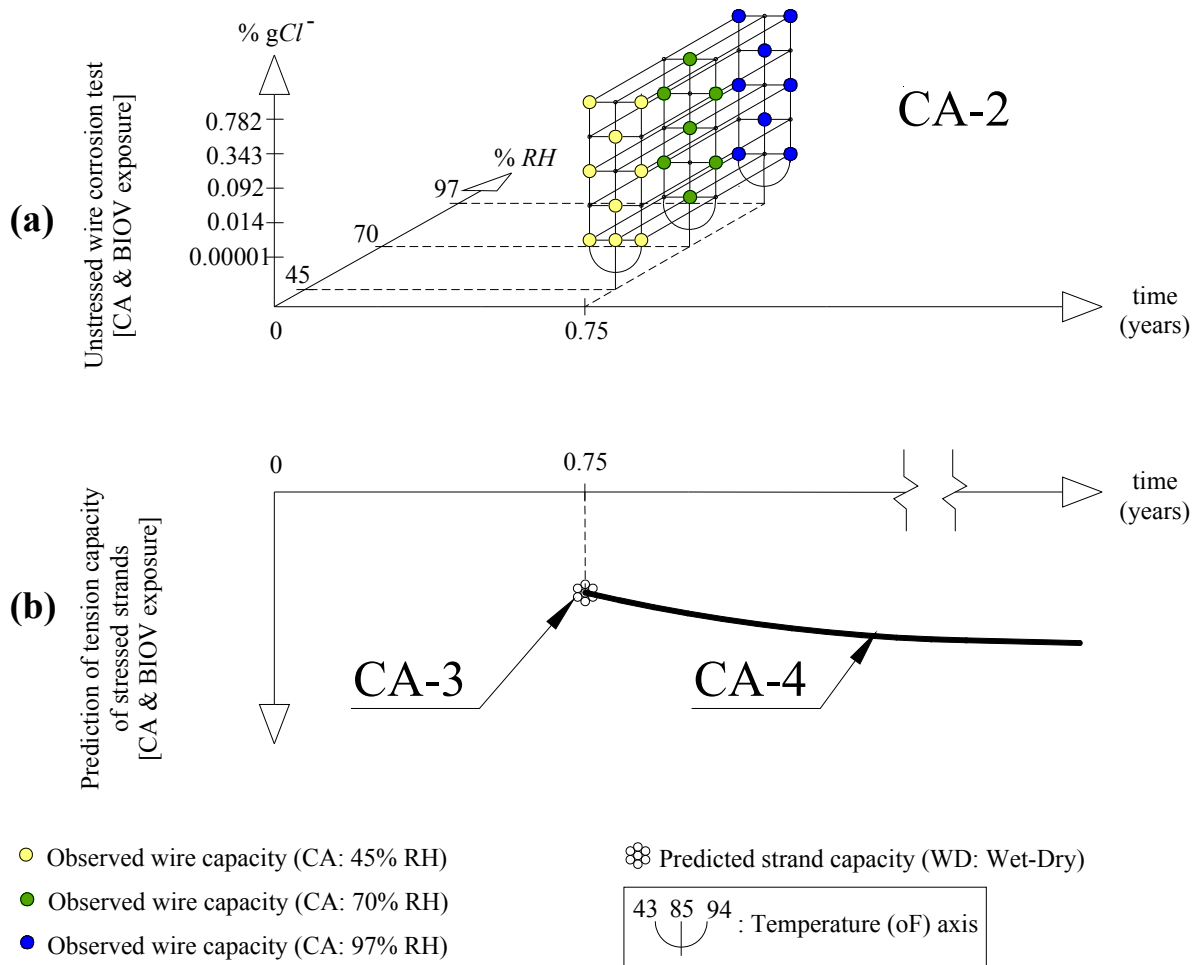


Figure 9-2. Graphical Representations of Steps CA-2, CA-3, and CA-4.

9.2.4. Assumptions for predicting strand capacity under CA conditions

Accurate estimation of the value of the power term, n_{CA} , is very critical to improve the accuracy of the predicted or extrapolated $C_{T,SS}$ at a future times. However, no long-term information could be obtained from long-term laboratory or field tests conducted as part of this study. Because of this, n_{CA} is assumed to be equal to -0.005 herein.

9.3. TENSION CAPACITY MODELS FOR STRANDS UNDER CA EXPOSURE CONDITIONS- RESULTS

This subsection develops the probabilistic model for the $C_{T,SS}$ under CA and BIOV exposure conditions based on the analytical steps that were outlined in Subsection 9.2.3.

9.3.1. Step CA-1: Develop probabilistic model for tension capacity of strands under CA and NV conditions

Negligible corrosion is expected when CA and NV exposure conditions exist. This indicates that $C_{T,SS}$ can be assumed to be equal to $C_{T,ARS}$. Hence, Equation (7.8) provided in Subsection 7.3 will be used to probabilistically predict $C_{T,SS}$ under CA and NV conditions.

9.3.2. Step CA-2: Develop probabilistic models for unstressed wires under CA and BIOV conditions

This subsection presents the diagnostic study on the data from wire samples exposed to CA conditions. Figure 9-3 shows the diagnostic plot showing the effect of % Cl^- on $C_{T,UW}$ at 9 months of CA exposure. Except the four data points inside the ellipse, no significant reduction in $C_{T,UW}$ has been observed. These four data points correspond to the extreme corrosive condition with 0.782 % Cl^- , 97 % RH , and 94°F (34.4°C). Figure 9-4 shows the diagnostic plot that illustrates the effect of % RH on $C_{T,UW}$ at 9-month CA exposure. Like Figure 9-3, Figure 9-4 also shows the four data points (inside the ellipse) with significant reduction in $C_{T,UW}$. In addition to this, a few data points near the top right corner (inside the small ellipse) indicate some reduction in $C_{T,UW}$ due to high % RH . Other than these four observations, all the data points show no significant reduction $C_{T,UW}$. Figure 9-5 shows the scatter plot showing the effect of T on the capacity of wires at 9-month CA exposure. The four data points corresponding to the extreme exposure condition with 0.782 % Cl^- , 97 % RH , and 94°F show significant reduction in capacity and are shown inside the ellipse near the bottom right corner. As observed in Figure 9-4, the data points inside the small ellipse near top right corner show some reduction in C_T and correspond to the condition with high T and high RH .

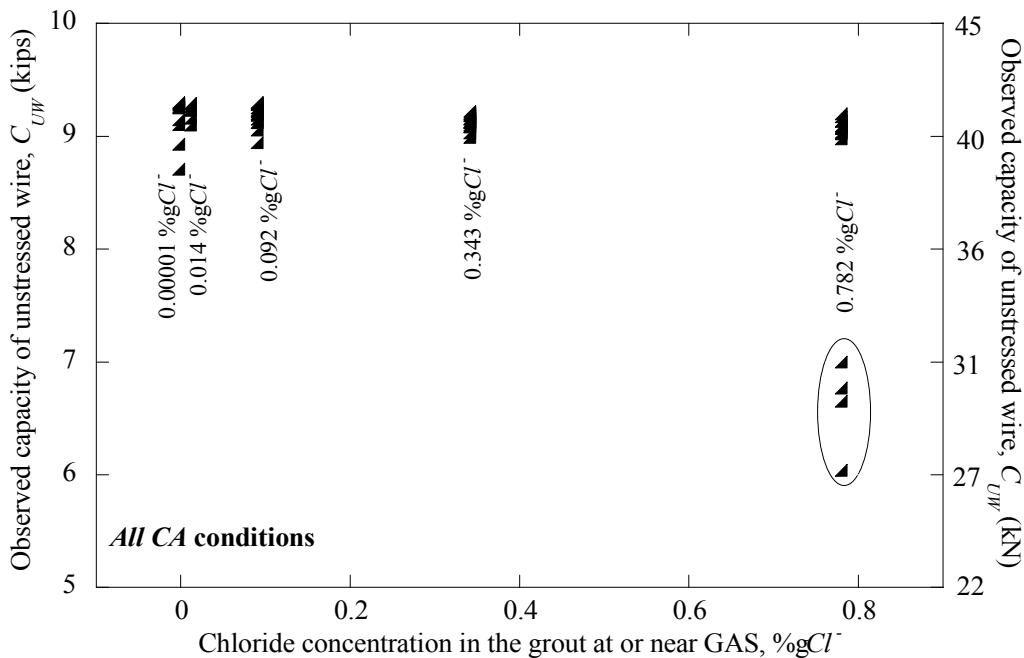


Figure 9-3. Scatter Plot between %gCl⁻ and Wire Capacity at 9-Month CA Exposure.

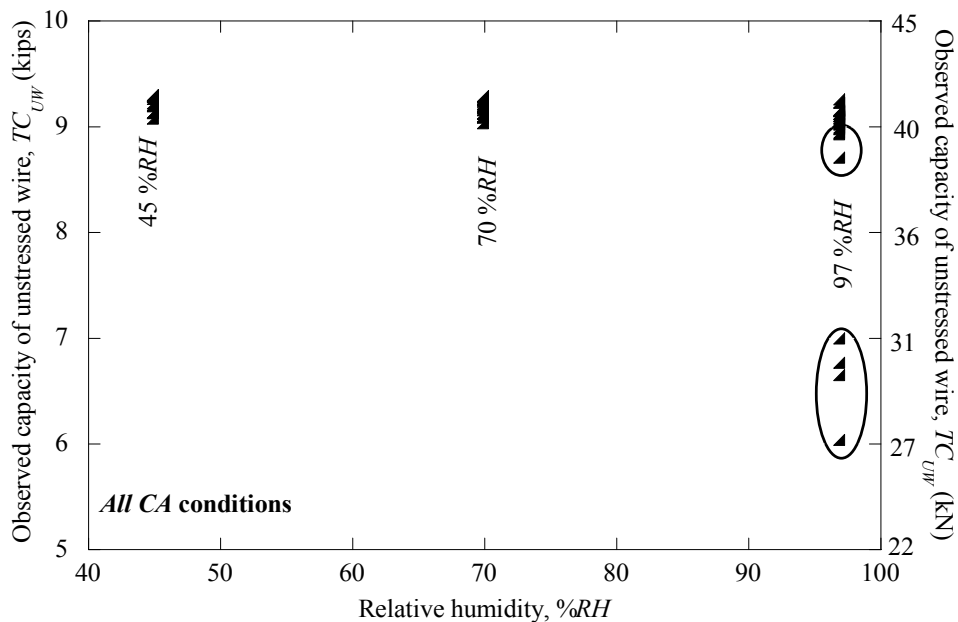


Figure 9-4. Scatter Plot between %RH and Wire Capacity at 9-Month CA Exposure.

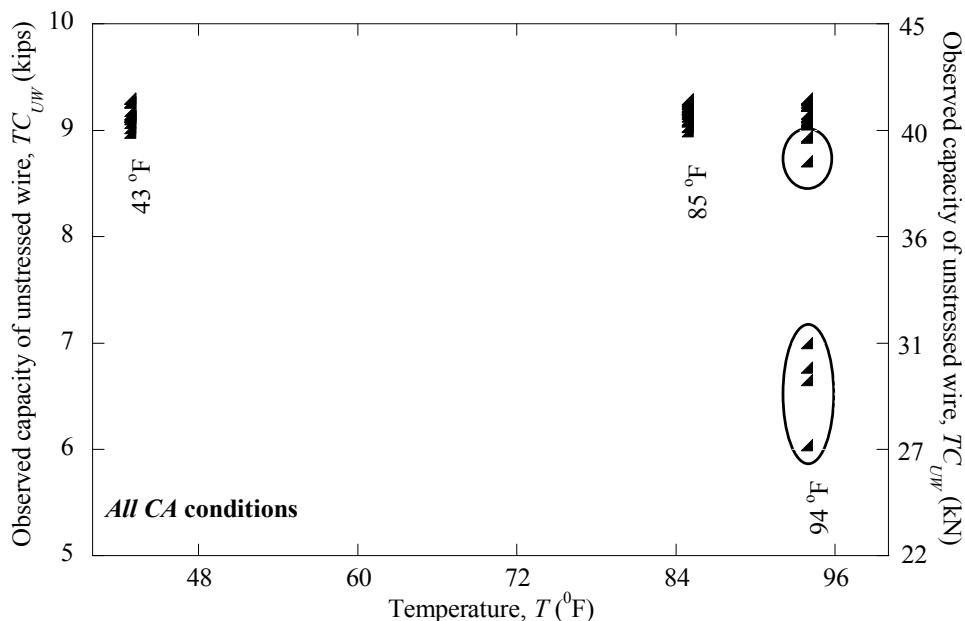


Figure 9-5. Scatter Plot between T and Wire Capacity at 9-Month CA Exposure.

These diagnostic plots indicate that an appropriate 3-way interaction term (between functions of $\%gCl^-$, $\%RH$, and $94^\circ F$) needs to be incorporated into the probabilistic model form. Figure 9-6 shows that the $\gamma_{\%gCl^-} \exp[\gamma_{RH}] \gamma_T$ term has an exponential effect on $C_{T,UW}$. Hence, this term could be incorporated into the model form. However, this 3-way interaction term cannot effectively capture the smaller reduction in $C_{T,UW}$ as exhibited by the data points inside the smaller ellipse on the top left corner in Figure 9-6. These data points correspond to 97%RH, $94^\circ F$, and 0.0001%gCl⁻. Literature (see Subsection 2.3.9) suggests that $\%RH$ has an exponential effect on the atmospheric corrosion process. Hence, the $\exp[\gamma_{RH}]$ term is also incorporated into the probabilistic model form.

Based on the diagnostic study and information from literature, the $UW_{CA, BIOV}$ model has been formulated as follows:

$$R_{C_{T,UW}}(\mathbf{x}, \Theta) \Big|_{t_{CA}=9 \text{ months}} = \theta_0 + \theta_1 \exp[\gamma_{RH}] + \theta_2 \exp[\gamma_{\%gCl^-} \exp[\gamma_{RH}] \gamma_T] + \sigma \varepsilon \quad (9.8)$$

where the terms are as defined in Equation (7.1) and Subsection 7.4.1. Note that this model form does not have a time function. A suitable form of time function will be discussed later. The full model form was assessed using the statistical procedures provided in Section 7.2.3. Table 9-1 provides the posterior statistics of the model.

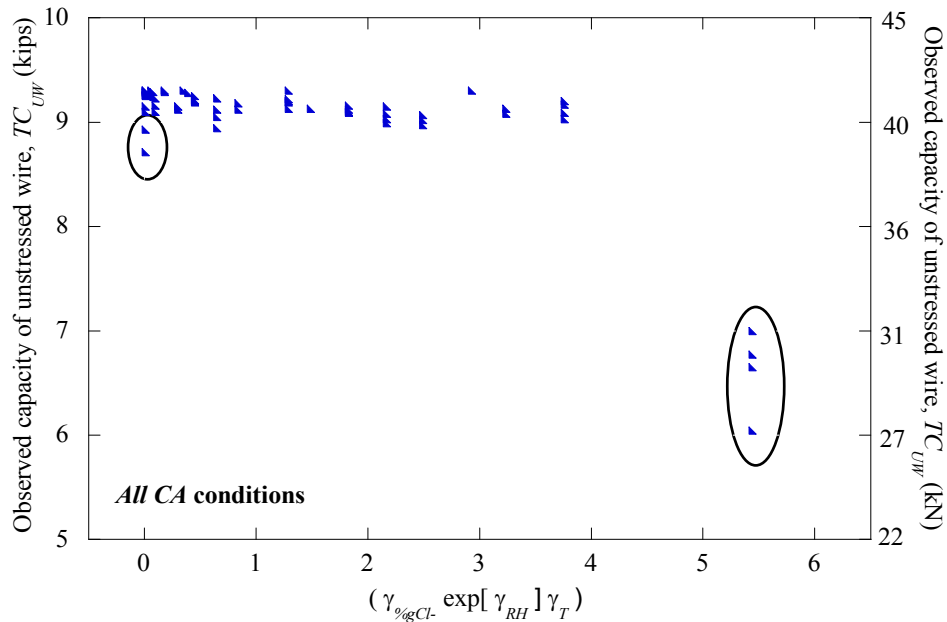


Figure 9-6. Scatter Plot between the 3-Way Interaction Term and Wire Capacity at 9-Month CA Exposure.

Table 9-1. MAPE and Posterior Statistics of UW_{CA, BIOV} Model.

Model Name	MAPE (%)	Parameters	Mean	Standard deviation	COV	Correlation coefficients between θ_i	
						θ_0	θ_2
UW _{CA, BIOV}	1.11	θ_0	0.1637	0.0018	0.01	1	
		θ_1	-0.0030	0.0008	-0.28	-0.98	1
		θ_2	-0.0002	0.0000	-0.04	0.20	-0.27
		σ	0.0027	0.0012			

The MAPE of the developed model is 1.11 percent. This very small MAPE does not necessarily mean that the model is extremely good. The MAPE is very small because a very small amount of corrosion was observed with most data points and the observed wire capacity exhibited very small scatter. Moreover, the COV of the $\exp[\gamma_{RH}]$ term is -0.28 , which is relatively moderate. Given the fact that most atmospheric corrosion models in literature accept variables with COVs as high as approximately 60 percent, the mean estimate of the coefficient of $\exp[\gamma_{RH}]$ term is acceptable. In addition, the COV of the coefficient corresponding to the $\gamma_{\%gCl^-} \exp[\gamma_{RH}] \gamma_T$ term is very small (i.e., -0.04), indicating good confidence in its mean estimate. Considering these values, the model form in Equation (9.8) with the statistical estimates in Table 9-1 provides reasonably good prediction, on the grounds of available experimental data. Because of these reasons, this model is referred to as Model $UW_{CA, BIOV}$ herein and can be expressed as follows:

$$R_{C_{T,UW}}(\mathbf{x}, \Theta) \Big|_{t_{CA}=9 \text{ months}} = 0.1637 - 0.0030 \exp[\gamma_{RH}] - 0.0002 \exp[\gamma_{\%gCl^-} \exp[\gamma_{RH}] \gamma_T] + 0.0027\epsilon \quad (9.9)$$

Figure 9-7 shows the validation plot for the $UW_{CA, BIOV}$ model. The details on analyzing validation plots are provided in Subsection 7.2.3.5. As is expected from a good capacity model, most data points are spread along the 1:1 line. Although a few data points near the top right corner lie outside the $\pm\sigma$ region, most other data points including the four data points near the lower left corner (i.e., those corresponding to the extreme exposure conditions) are adequately predicted. In general, the $UW_{CA, BIOV}$ model seems to provide good prediction of $C_{T,UW}$ under CA exposure conditions.

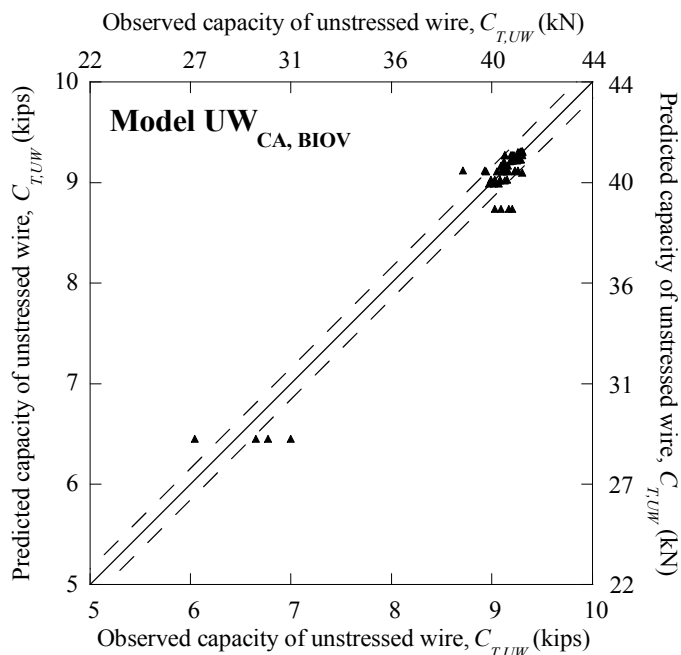


Figure 9-7. Validation Plot for the UW_{CA, BIOV} Model.

9.3.3. Step CA-3: Predict the capacity of stressed strands at 9 months

$C_{T,SS}$ at 9-month exposure to CA and BIOV conditions can be predicted by nesting the UW_{CA, BIOV} model (developed in Subsection 9.3.2) into the UW-SS_{BIOV} model (developed in Subsection 8.3.4). Based on the definition in Subsection 8.3.4, the UW-SS_{BIOV} model is:

$$R_{C_{T,SS}} \Big|_{t_{CA}=9 \text{ months}} = \theta_0 \left(\gamma_{C_{T,UW}} \Big|_{t_{CA}=9 \text{ months}} \right)^{\theta_1} + \sigma \epsilon = 7.7492 \left(\gamma_{C_{T,UW}} \Big|_{t_{CA}=9 \text{ months}} \right)^{1.0924} + 0.0619\epsilon \tag{9.10}$$

By substituting the term $\gamma_{C_{T,UW}} \Big|_{t_{CA}=9 \text{ months}}$ in Equation (9.10) with the right-hand side of Equation (9.9), we get:

$$R_{C_{T,SS}} \Big|_{t_{CA}=9 \text{ months}} = 7.7492 \left(0.1637 - 0.0030 \exp[\gamma_{RH}] - 0.0002 \exp[\gamma_{\%gCl^-} \exp[\gamma_{RH}] \gamma_T] + 0.0027 \varepsilon \right)^{1.0924} + 0.0619 \varepsilon \quad (9.11)$$

In Equation (9.11) the standard normal variables (i.e., ε) appear in two places. Because these two terms are correlated, Equation (9.11) is simplified by removing the inner error term (i.e., 0.0027ε). This will avoid the complexities associated with correlated random variables. The reduced expression is as follows:

$$R_{C_{T,SS}} \Big|_{t_{CA}=9 \text{ months}} = 7.7492 \left(0.1637 - 0.0030 \exp[\gamma_{RH}] - 0.0002 \exp[\gamma_{\%gCl^-} \exp[\gamma_{RH}] \gamma_T] \right)^{1.0924} + 0.0619 \varepsilon \quad (9.12)$$

Equation (9.12) can be used to predict $C_{T,SS}$ at t_{CA} equal to 9 months.

9.3.4. Step CA-4: Develop the model for stressed strands under CA and BIOV exposure conditions

The value of $C_{T,SS}$ at any time, t_{CA} , can be predicted by incorporating Equation (9.12) into a power model form as follows:

$$R_{C_{T,SS}} = R_{C_{T,SS}} \Big|_{t_{CA}=9 \text{ months}} \times \left(\frac{t_{CA}}{0.75} \right)^{n_{CA}} + \sigma \varepsilon; \quad 9 \text{ months} = 0.75 \text{ years} \quad (9.13)$$

where t_{CA} is the total time of CA exposure (in years) and n_{CA} is a constant. The model error term (i.e., $\sigma \varepsilon$) is added after the extrapolation process using the power model form. In this way, the substitution of Equation (9.12) into Equation (9.13) results in:

$$R_{C_{T,SS}} = \left(7.7492 \left(0.1637 - 0.0030 \exp[\gamma_{RH}] - 0.0002 \exp[\gamma_{\%gCl^-} \exp[\gamma_{RH}] \gamma_T] \right)^{1.0924} \right) \times \left(\frac{t_{CA}}{0.75} \right)^{n_{CA}} + 0.0619\varepsilon \quad (9.14)$$

Equation (9.14) is defined as Model $SS_{CA, BIOV}$ herein and can be used to predict $C_{T,SS}$ under CA and $BIOV$ conditions.

9.3.5. Prediction of tension capacity of stressed strands under CA and $BIOV$ conditions

The prediction of C_T using the developed models is heavily dependent on the value of n_{CA} . Ideally, the value of n_{CA} should be obtained based on the calibration of the developed models with the field data. Such field data can be obtained from an investigation of corrosion on PT strands exposed to CA conditions for long exposure periods (say, decades).

In this subsection, this study assumed n_{CA} to be equal to -0.005 and predicted $C_{T,SS}$ values using the $SS_{CA, BIOV}$ model. Figure 9-8(a), (b), (c), and (d) show the predicted $C_{T,SS}$ values corresponding to $\%gCl^-$ values of 0.014, 0.092, 0.343, and 0.782 $\%gCl^-$, respectively. The horizontal dashed lines in Figure 9-8 indicate the nominal capacity of strands (i.e., $MUTS_{ARS}$) that is 58.6 kips (261 kN). Extrapolated values of $C_{T,SS}$ corresponding to the following four combinations of RH and T are shown.

1. an extreme CA condition with 90 % RH and 90°F (32.2°C),
2. a moderate CA condition with 70 % RH and 90°F (32.2°C),
3. a normal CA condition with 70 % RH and 60°F (15.6°C), and
4. a normal CA condition with 70 % RH and 70°F (21.1°C).

A general conclusion is drawn that synergistic occurrences of high $RH-T$ or high $RH-T-\%gCl^-$ levels can cause severe localized atmospheric corrosion and capacity loss in a very short period of time compared to the design life of the bridge.

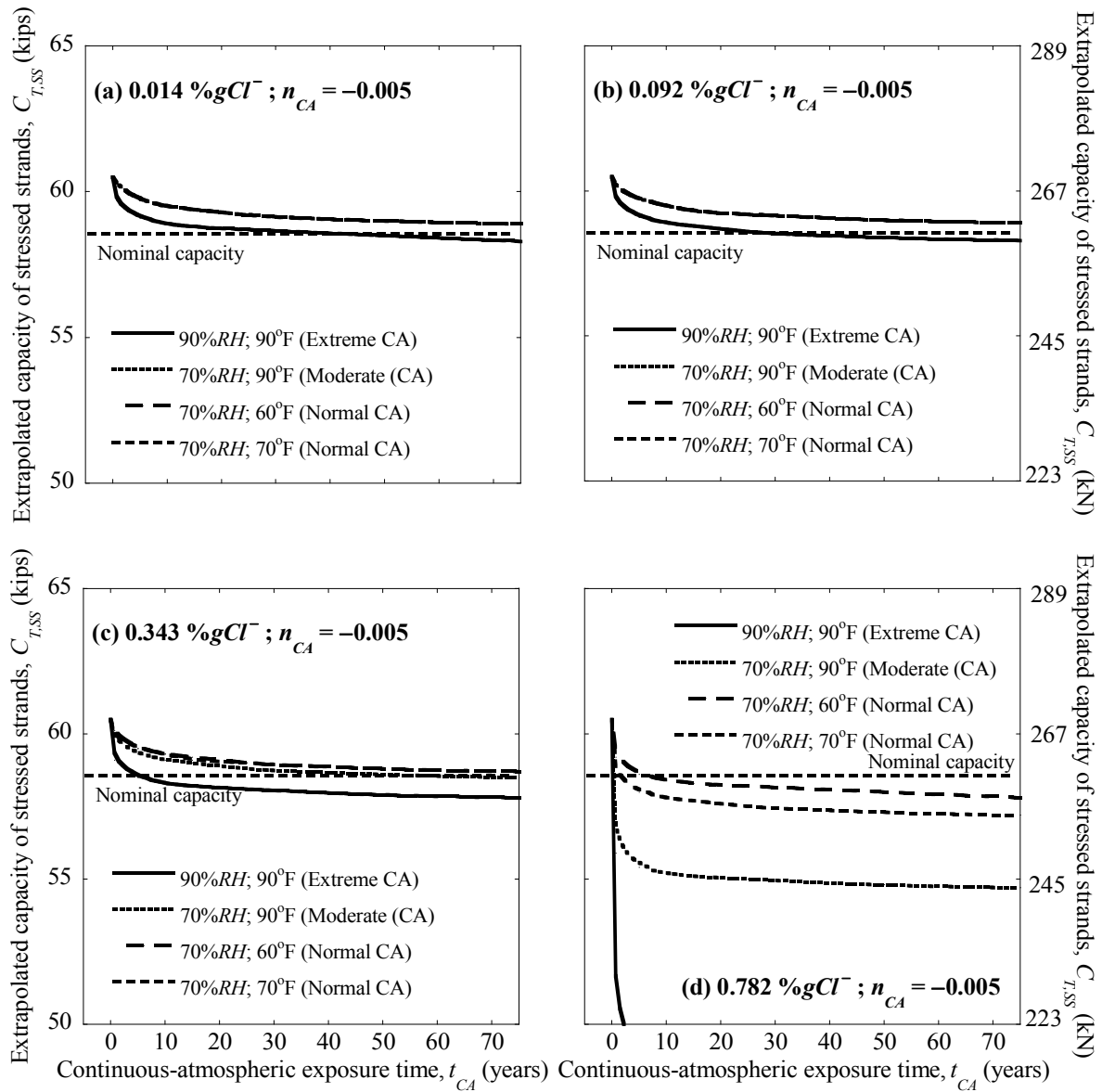


Figure 9-8. Capacity of Stressed Strands under CA Conditions with $n_{CA} = -0.005$.

9.4. SUMMARY

In this section, a probabilistic model for C_T of stressed strands under CA exposure was developed using the stressed strand–unstressed wire relationships (from Section 8) and

experimental data on unstressed wires (from Subsection 6.4.3.4). The probabilistic models developed in Sections 7 and 9 will be used appropriately in the next section on modeling and assessment of structural reliability of segmental concrete bridges with voided PT systems.

10. MODELING AND ASSESSMENT OF STRUCTURAL RELIABILITY OF SEGMENTAL BRIDGES

Equation Section (Next)

10.1. INTRODUCTION AND OBJECTIVE

This section presents the modeling and assessment of the structural reliability of PT bridges. First, the objectives of this section are presented. Then a framework to determine the generalized reliability index, β , is presented. Following this, the strength reliability index, $\beta_{strength}$, is formulated as a function of moment capacity (C_M) and moment demand (D_M). The algorithms for determining C_M and D_M are then developed. The service reliability index, $\beta_{service}$, is then formulated as a function of stress capacity (C_f) and stress demand (D_f). The algorithms for determining C_f and D_f are then developed using the existing formulations for allowable and applied flexural stresses at girder midspan. Following this, the statistical models of all the random variables are provided. A typical PT bridge is then defined on the basis of geometrical and structural characteristics. As an application of the developed reliability models, time-variant $\beta_{strength}$ and $\beta_{service}$ (for a period of 75 years) for this typical PT bridge are determined based on a pre-defined set of constant and random parameters.

The objective of this section is to model and assess the time-variant strength and service reliabilities of a typical PT bridge experiencing HS20 and HL93 loading conditions.

10.2. A FRAMEWORK TO DETERMINE STRUCTURAL RELIABILITY

Figure 10-1 shows a simplified flowchart of the process used in this research to determine the generalized reliability index, β , of PT bridges. In Figure 10-1, the box with a thick border indicates the Monte Carlo simulation process used to determine P_f with an acceptable level of accuracy ($COV(P_f) \leq COV(P_f)_{target}$) and β . The two shaded boxes in Figure 10-1 indicate the definitions of the random parameters and the $COV(P_f)_{target}$ and the evaluation of the limit state function, g .

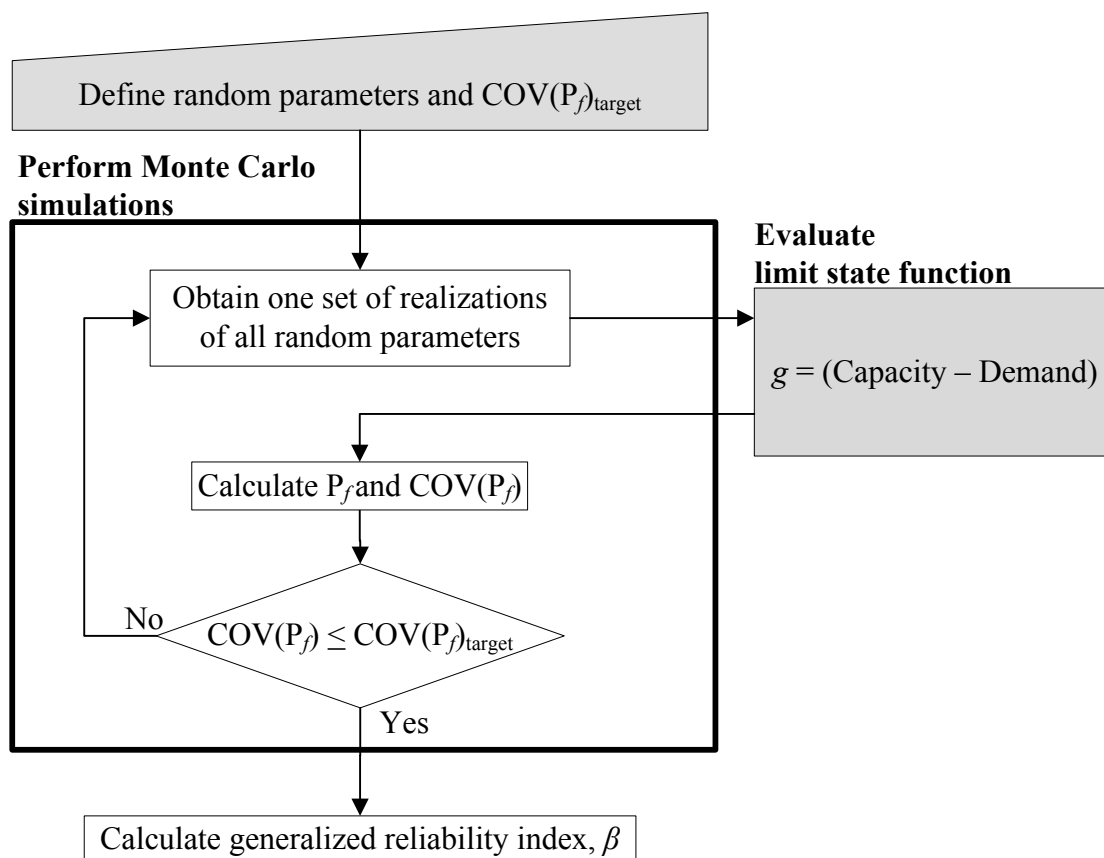


Figure 10-1. A Framework to Model and Assess the Generalized Reliability Index, β .

Formulations to evaluate g , P_f , $\text{COV}(P_f)$, and β are available in literature and were reviewed in Subsection 2.9.2. The literature provides analytical formulations to estimate g for strength and service limit states (i.e., $g_{strength}$ and $g_{service}$) for pre-tensioned structural systems with “as-received” strands. However, these models do not capture the potential reduction in $\beta_{strength}$ and $\beta_{service}$ due to the corrosion and resulting reduction in the C_T of strands as a function of time.

In the following subsections, $g_{strength}$ and $g_{service}$ are formulated based on capacity and demand functions. Various parameter uncertainties are taken into account while formulating these capacity and demand functions. The uncertainties associated with the tension capacity (C_T) and prestress loss (P_{loss}) of individual strands, compressive strength of concrete (f_c), void condition (VC) in individual external ducts, damage conditions (DC) of individual external ducts, and elements of applied dead load (DL) and live load (LL) are considered. The analytical formulations for the capacity and demand functions for $g_{strength}$ and $g_{service}$ are then programmed

using MATLAB[®] and incorporated into FERUM (discussed in Subsection 2.9.2.3) to determine $\beta_{strength}$ and $\beta_{service}$.

10.3. MODELING STRENGTH RELIABILITY

10.3.1. Strength limit state function, probability of strength failure, and strength reliability index

A review on limit state function, probability of failure, and reliability indices was provided in Subsection 2.9.2.1. Following the conventional structural reliability theory (Ditlevsen and Madsen 1996), the strength limit state function, $g_{strength}$, is defined as follows:

$$g_{strength} = C_M - D_M \quad (10.1)$$

such that the events with $g_{strength} \leq 0$ represent strength failure. Based on Monte Carlo simulations of $g_{strength}$, the probability of strength failure, $P_{f, strength}$, can be determined as follows:

$$P_{f, strength} = \frac{N_{f, strength}}{N_{sim, strength}} \quad (10.2)$$

where $N_{f, strength}$ and $N_{sim, strength}$ indicate the number of instances with strength failure and number of simulations, respectively. The value of $COV(P_{f, strength})$ is determined as follows (Haldar and Mahadevan 2000):

$$COV(P_{f, strength}) \approx \frac{1}{P_{f, strength}} \cdot \sqrt{\frac{(1 - P_{f, strength}) P_{f, strength}}{N_{sim, strength}}} \quad (10.3)$$

Based on the estimated $P_{f, strength}$, the generalized strength reliability index, $\beta_{strength}$, can be determined as follows (Ditlevsen and Madsen 1996):

$$\beta_{strength} = -\Phi^{-1}(P_{f, strength}) \quad (10.4)$$

where Φ is the cumulative distribution function (CDF) of the standard normal distribution. To determine $\beta_{strength}$, C_M and D_M need to be modeled.

10.3.2. Probabilistic modeling of moment capacity of the girder at mid-span

Ting and Nowak (1991) developed a computer algorithm to study the effect of decrease in C_T of strands on C_M of pre-tensioned beams. Ting and Nowak (1991) assumed constant initial prestress force, P_i , loss in prestress force, P_{loss} , and loss in C_T , for all the strands at the same depth in a beam. Depending on the uncertainties associated with the stress-strain and corrosion mechanisms, this might not be the case in segmental, PT bridges.

AASHTO (1999) and AASHTO (2007) provide guidance to design segmental bridges. These specifications were calibrated to design new bridges with a target reliability index, β_{target} , value of 3.5 (Nowak and Collins 2000). Not all formulations in these design documents can be used for assessing C_M of existing segmental bridges, especially when uncertainties exist in capacity and demand parameters. A probabilistic approach is needed to determine C_M of segmental, PT bridges.

This research developed an algorithm to determine the probabilistic C_M of PT bridges. The algorithm to determine C_M uses the probabilistic models for strand capacity developed in this research, the formulations for stress and strain in concrete from Todeschini et al. (1964), and the formulations for nominal tensile stress of unbounded strands from AASHTO (2007) specifications. The non-linear model by Todeschini et al. (1964) was selected because it is a single closed-form solution and, hence, suitable for efficient numerical simulation.

Figure 10-2 shows a general schematic of strain, stress, and force distributions on the cross section of a typical T-girder.

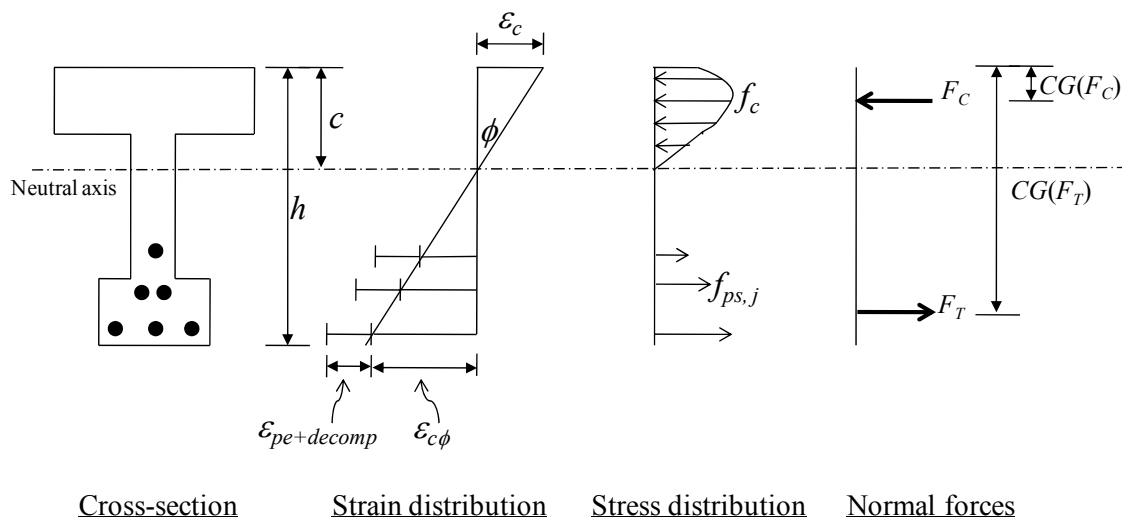


Figure 10-2. Strain, Stress, and Normal Force Distributions on a T-girder.

In the strain diagram, the $\varepsilon_{pe+decomp}$ is defined as the the total tensile strain on strands when concrete decompression begins. In this research, this strain is assumed to be equal to the strain induced by the total effective prestress, f_{pe} , when concrete decompression begins. Note that in this research, f_{pe} is calculated by subtracting prestress losses at a given time due to long-term effects, Δf_{pLT} , from the initial prestress after anchoring, f_{pi} (see Subsection 2.7.1.2 for further details). The value of $\varepsilon_{pe+decomp}$ for each strand may be different due to the uncertainty in f_{pe} . The total stress in each strand, $f_{ps,j}$ (j indicating j^{th} strand), is calculated using the formulation in AASHTO (2007) specifications. The compressive strain in concrete, ε_c , is assumed linear and can be calculated using the value of c and ϕ . The compressive stress in concrete, f_c , is calculated using the nonlinear stress-strain model from Todeschini (1964). In the force diagram, the total compressive force in concrete (F_C) and tensile force in strands (F_T) and corresponding center of gravities are shown. Note that PT bridges are typically under compression when all the strands are intact and under service loads. However, when strands corrode, the neutral axis can move upward resulting in the reduction in the compressive forces at lower fibers in the cross-section and potentially leading to a net tensile force.

C_M is determined considering tension failure of individual strands and compression failure of concrete. This algorithm to determine C_M is structured using a main function and two subfunctions as follows:

- a main function to determine C_M ;
- a subfunction to determine the bending moment, M_ϕ , corresponding to any particular value of curvature, ϕ . In the following discussion, the flowchart of this subfunction is denoted as “ M ”; and
- a subfunction to determine total normal tensile force, F_T . In the following discussion, the flowchart of this subfunction is denoted as “ F_T .”

10.3.2.1. The main function to determine moment capacity

The algorithm (see Figure 10-3 for flowchart) to determine the probabilistic C_M is as follows:

1. Determine the dimensions of the effective cross-section using the AASHTO (2007) specification for the effective flange width for computing nominal moment. Divide the effective cross-section into convenient rectangular or triangular subsections. Also, assume that the two web sections are vertical. Now, obtain sectional properties (area, moment of inertia, location, etc.) of each subsection.
2. Obtain the configuration and location of each PT strand. Calculate the effective prestress force, $P_{e,j}$, in each strand as follows:

$$P_{e,j} = P_i - P_{loss,j} = (0.70 \times MUTS_{ARS}) - P_{loss,j} \quad (10.5)$$

where the subscript j indicates the j^{th} strand, P_i is the average prestress force after anchoring, and $P_{loss,j}$ is the loss in prestress.

3. Determine the environmental, void, and damage conditions. Following the strategy provided later in Subsection 10.5.2, select the probabilistic model for C_T of each strand. Calculate the $C_{T,j}$ for each strand. Calculate the remaining area of each strand, $A_{ps,j}$, as follows:

$$A_{ps,j} = C_{T,j} \left(\frac{A_{ARS}}{C_{T,ARS}} \right) \quad (10.6)$$

- where the subscript j indicates the j^{th} strand, and A_{ARS} and $C_{T,ARS}$ are the cross-sectional area and average tension capacity of “as-received” strands with negligible corrosion.
4. Set $\phi_{\min} = 0$, $\phi_{\max} = 0.01$, and $\delta_{\phi} = 1 \times 10^{-6}$ radians/inch.
 5. Set the curvature, $\phi = (\phi_{\min} + \phi_{\max})/2$. Also, set $\phi_{+} = (\phi + \delta_{\phi})$ and $\phi_{-} = (\phi - \delta_{\phi})$. [Note: These points are moved to either leftward or rightward based on a decision process as described later in step 8. The positions of ϕ , ϕ_{+} , and ϕ_{-} during the first iteration are shown in Figure 10-4.]
 6. Call the subfunction to calculate bending moment (This subfunction is presented in 10.3.2.2). Calculate bending moment corresponding to ϕ , ϕ_{+} , and ϕ_{-} (i.e., M_{ϕ} , $M_{\phi_{+}}$, and $M_{\phi_{-}}$, respectively).
 7. Determine the slope of moment-curvature curve before and after ϕ (i.e., $S_{\phi_{-}}$ and $S_{\phi_{+}}$). [Note: $S_{\phi_{-}}$ and $S_{\phi_{+}}$ during the first iteration are shown in Figure 10-4]. Go to step 8.
 8. If $S_{\phi_{+}} > 0$, then set $\phi_{\min} = \phi$ to move the ϕ rightward and repeat steps 5 through 7. If $S_{\phi_{-}} \leq 0$ and $S_{\phi_{+}} \leq 0$, then set $\phi_{\max} = \phi$ to move the ϕ leftward and repeat steps 5 through 7. [Note: Possible positions of ϕ , ϕ_{+} , and ϕ_{-} in the second iteration (i.e., after moving either leftward or rightward) are shown in Figure 10-4. [Note: Figure 10-4 does not show these positions in further iterative processes or until a solution is obtained.] If $S_{\phi_{-}} > 0$ and $S_{\phi_{+}} < 0$, then define C_M equal to M_{ϕ} . [Note: The “peak point” in Figure 10-4 corresponds to C_M .]

In each iteration, step 6 of the main function determines ϕ , ϕ_{+} , and ϕ_{-} and calls the subfunction M to calculate M_{ϕ} , $M_{\phi_{+}}$, and $M_{\phi_{-}}$. The details of the subfunction M are discussed next.

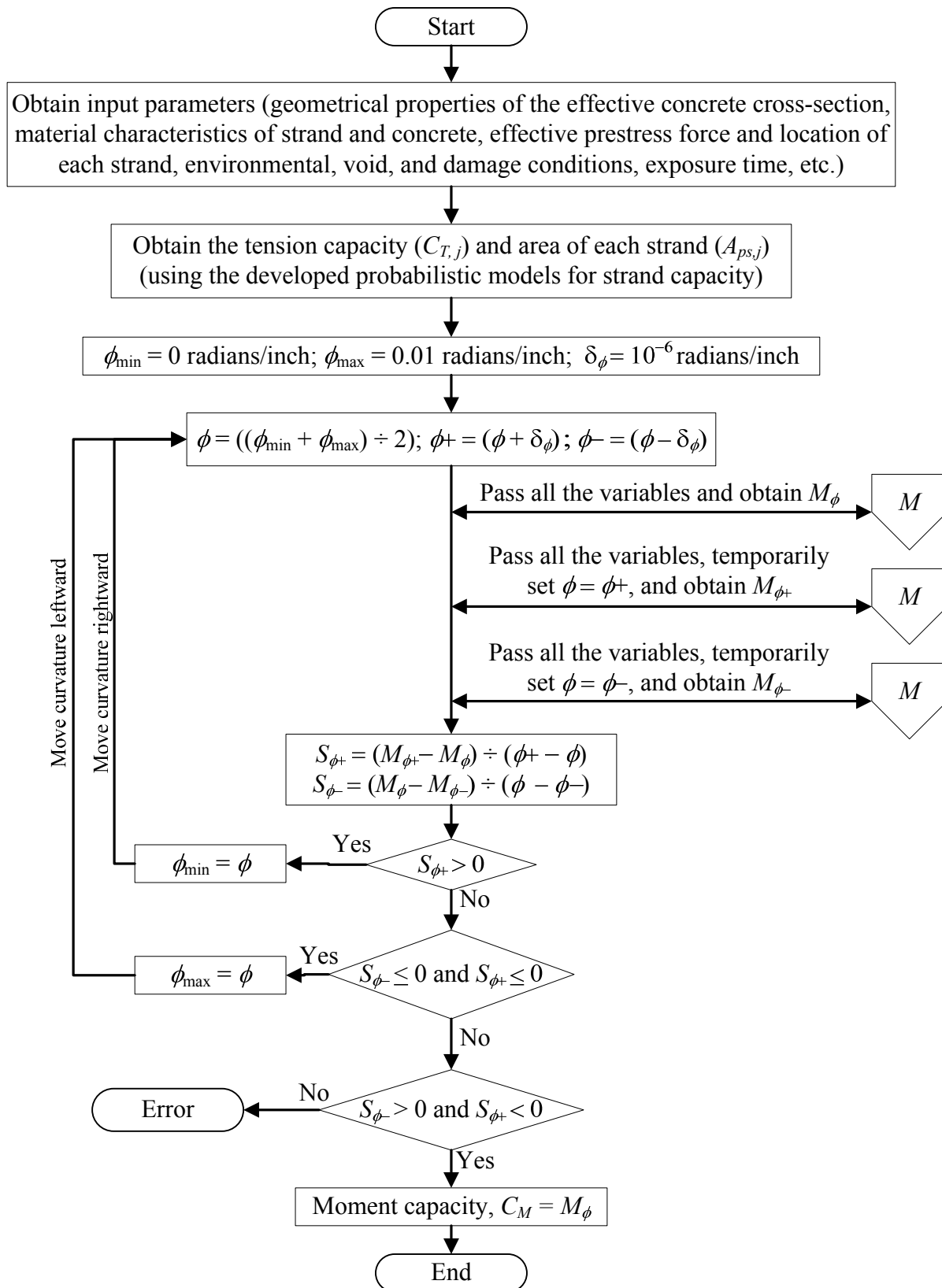


Figure 10-3. Simplified Flowchart to Determine Moment Capacity of a PT Girder.

3.

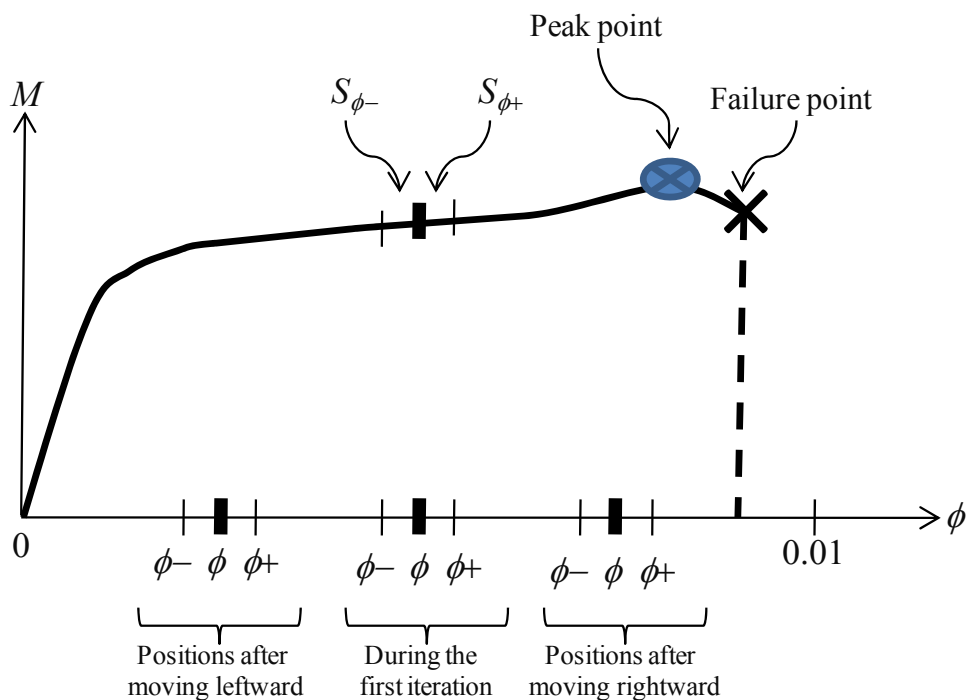


Figure 10-4. Schematic Showing the Movement of Curvature, ϕ .

10.3.2.2. The subfunction to determine bending moment corresponding to a curvature

Within one iteration, step 6 of the main function for calculating C_M calls the subfunction to calculate M_ϕ , $M_{\phi+}$, and $M_{\phi-}$. The algorithm to calculate M at any value of ϕ is as follows (see Figure 10-5 for flowchart):

1. Obtain the value of curvature, ϕ , and other parameters from the calling function.
2. Set $c_{\min} = 0$ and $c_{\max} =$ overall height of the girder, h . Note that all the distances are measured from top of the girder.
3. Set the neutral axis depth, $c = (c_{\min} + c_{\max})/2$.

4. If the strain at the extreme compression fiber ($c\phi$) is greater than 0.003 (the maximum usable concrete compressive strain defined by AASHTO [2007]), then set $c_{\max} = c$ to move the neutral axis upward. Go to step 3.
5. Call the subfunction to calculate the total normal tensile force, F_T , and its center of gravity, $CG(F_T)$. This subfunction is presented in 10.3.2.3.
6. Assume a linear strain distribution and use the non-linear stress model by Todeschini et al. (1964) to calculate the compression force in each subsection, $F_{C,k}$ (the subscript k indicates k^{th} “convenient rectangular or triangular subsection” discussed in the step 1 of the main function). This is done by integrating the estimated concrete compressive stress f_c over the area of each of these subsections. Then, calculate the total compression force, F_C , and its center of gravity, $CG(F_T)$.
7. If $(|F_T| - |F_C|) > 0.001 \times |F_C|$, then force equilibrium does not exist. If $|F_C| > |F_T|$, then set $c_{\max} = c$ to move the neutral axis upward. Go to step 3. If $|F_C| < |F_T|$, then set $c_{\min} = c$ to move the neutral axis downward. Go to step 3.
8. If $(|F_T| - |F_C|) \leq 0.001 \times |F_C|$, then force equilibrium exists. Calculate average normal force, $F = (|F_T| + |F_C|)/2$. Calculate moment arm, $k = CG(F_T) - CG(F_C)$. Calculate bending moment, M , by multiplying F and k . Return the value of M to the calling function.

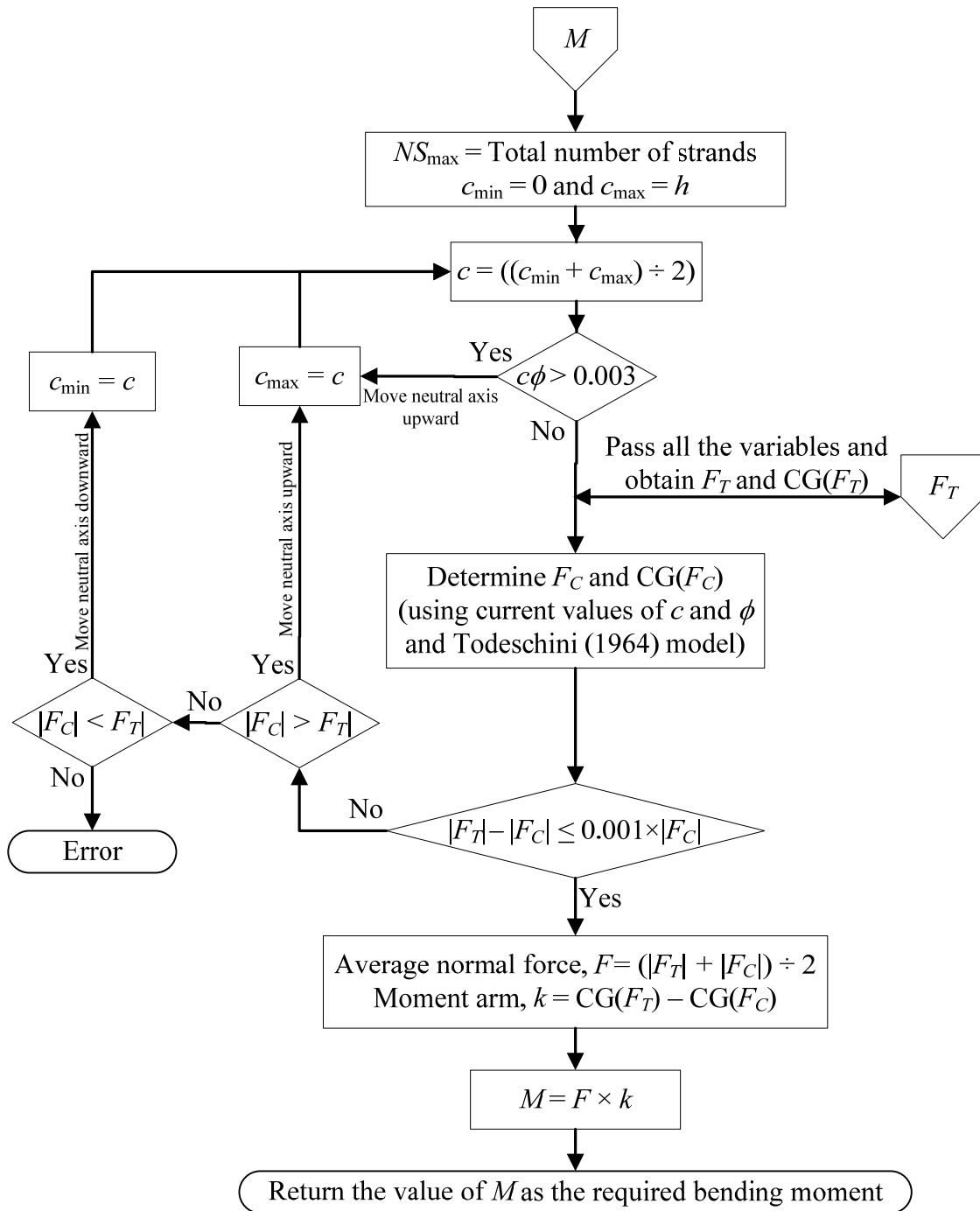


Figure 10-5. Simplified Flowchart to Determine Moment Corresponding to a Curvature.

10.3.2.3. The subfunction to determine the total normal tensile forces

Step 5 in the subfunction to calculate bending moment, M , calls the subfunction to calculate F_T . The algorithm (see Figure 10-6 for flowchart) for this subfunction to calculate F_T is provided here.

1. Obtain all the parameters from the calling subfunction, M .
2. Obtain the count for each strand, $j = 1$.
3. If $A_{ps,j}$ is 0, then set $f_{pe,j}$, $f_{ps,j}$, and $P_{e,j}$ to 0. Go to step 5.
4. If $A_{ps,j}$ is greater than 0, calculate the average stress, $f_{ps,j}$ (ksi), in each strand at nominal conditions using Equation (10.7) for unbonded prestressing strands, provided in AASHTO (2007).

$$5. \quad f_{ps,j} = \underbrace{\left[f_{pe,j} + 900 \left(\frac{d_{p,j} - c}{l_e} \right) \right]}_{f_{ps,j, \text{calculated}}} \leq f_{py}; \quad l_e = \left(\frac{2l_i}{2 + N_s} \right) \quad (10.7)$$

where $f_{pe,j}$ (ksi) is the effective prestress in the j^{th} strand, $d_{p,j}$ is the distance between the extreme compression fiber and centroid of the j^{th} strand (inches), l_e is the effective tendon length (inches), l_i is the length of the strand between anchorages (inches), N_s is the number of support hinges crossed by the strand between the anchorages, and f_{py} is the yield strength of strand (ksi). The value of $f_{ps,j, \text{calculated}}$ varies due to the variation in c . If $f_{ps,j, \text{calculated}}$ is greater than the ultimate tensile stress capacity of strand, f_{pu} , then the strand fails in tension and is removed from further calculations by setting its $f_{ps,j}$, $f_{pe,j}$, $P_{e,j}$, and $A_{ps,j}$ to 0. If $f_{ps,j, \text{calculated}}$ is greater than 0 and less than f_{py} , then set $f_{ps,j}$ is equal to $f_{ps,j, \text{calculated}}$.

6. Calculate $F_{T,j}$ for the unbroken strands by multiplying corresponding $A_{ps,j}$ with $f_{ps,j}$. Go to step 3 until $F_{T,j}$ for all the strands are determined (i.e., j reaches $N_{strands}$).
7. Determine the total tensile force, F_T , by adding the individual tensile forces, $F_{T,j}$. Determine the center of gravity of F_T , $CG(F_T)$. Note that this is measured from the top of the girder. See Figure 10-2 for graphical representation of F_T and $CG(F_T)$. Return the value of F_T and $CG(F_T)$ to the calling function.

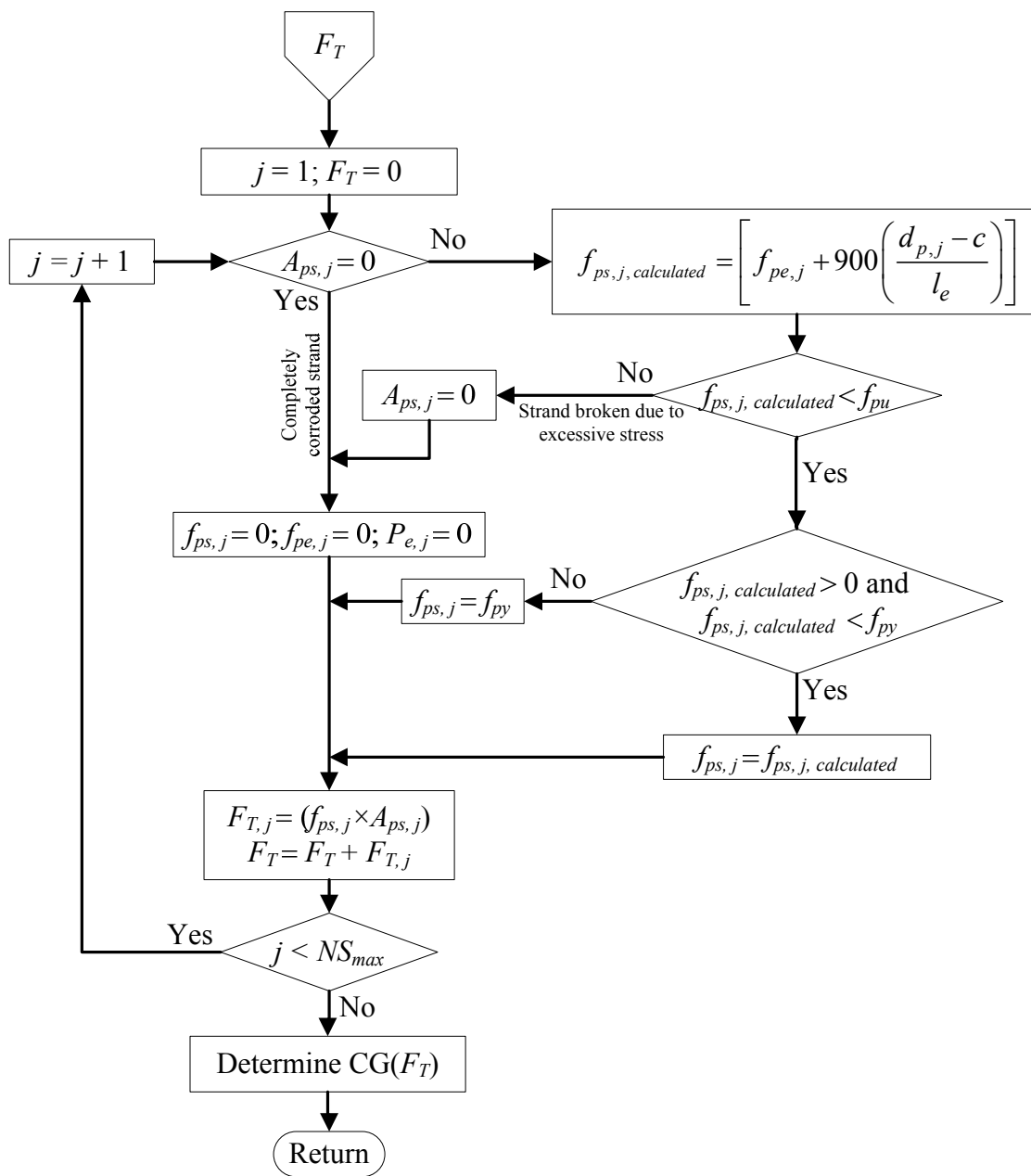


Figure 10-6. Simplified Flowchart to Determine the Total Normal Tensile Force, F_T .

An algorithm to determine C_M of a segmental, PT bridge with unbonded tendons that are exposed to various corrosive conditions is now developed.

10.3.3. Probabilistic modeling of moment demand on the girder at midspan

Dead, live, and impact loads are used in modeling D_M . Uniformly distributed dead load can be calculated using the unit weight and volume of the reinforced concrete (or other materials) used in the box section, overlay and future wearing surface, and side barriers. The permanent dead load due to the weight of box section is denoted as DL_1 . The total weight of the overlay and future wearing surface and side barriers is denoted as DL_2 . Following the procedures in AASHTO (1996) and AASHTO (2007), the design lane (LL_{lane}), truck (LL_{truck}), and tandem (LL_{tandem}) loads are used to calculate the total live load for HS20 and HL93 loading conditions. Per AASHTO (1996), an impact load equal to $(50 / [125 + \text{span in ft}])$ percent of live load is included for HS20 loading. Per AASHTO (2007), an impact load equal to 33 percent of live load is included for HL93 loading. Using structural mechanics principles and influence line theory, the critical section with the maximum moment is determined for the simply supported segmental girder. D_M is defined to be equal to this maximum moment. The uncertainty in D_M is captured using the random parameters for DL_1 , DL_2 , LL_{lane} , LL_{truck} , and LL_{tandem} .

10.4. MODELING SERVICE RELIABILITY

10.4.1. Service limit state function, probability of service failure, and service reliability

index

A review on g , P_f , and β was provided in Subsection 2.9.2.1. Following the conventional structural reliability theory (Ditlevsen and Madsen 1996), the service limit state function, $g_{service}$, is defined as follows:

$$g_{service} < 0 \left\{ \begin{array}{l} g_{f,top} < 0 \\ \text{or} \\ g_{f,bottom} < 0 \end{array} \right. \quad (10.8)$$

where,

$$g_{f,top} = C_{f,top} - D_{f,top}$$

$$g_{f,bottom} = (-C_{f,bottom}) - (-D_{f,bottom})$$

where $g_{f, top}$ and $g_{f, bottom}$ are defined as limit state functions for stresses at the extreme top and bottom fibers, respectively; $C_{f, top}$ and $C_{f, bottom}$ represent the allowable stresses (or stress capacities) at extreme top and bottom fibers, respectively; and $D_{f, top}$ and $D_{f, bottom}$ represent the corresponding applied stresses (stress demands). The events $g_{service} \leq 0$ represent the service failure. As for $P_{f, strength}$, based on Monte Carlo simulations of $g_{service}$, the probability of service failure, $P_{f, service}$, can be determined as follows:

$$P_{f, service} = \frac{N_{f, service}}{N_{sim, service}} \quad (10.9)$$

where $N_{f, service}$ and $N_{sim, service}$ indicate the number of instances with service failure and number of Monte Carlo simulations, respectively. The $COV(P_{f, service})$ is determined as follows (Haldar and Mahadevan 2000):

$$COV(P_{f, service}) \approx \frac{1}{P_{f, service}} \cdot \sqrt{\frac{(1 - P_{f, service}) P_{f, service}}{N_{sim, service}}} \quad (10.10)$$

Based on the estimated $P_{f, service}$, the generalized strength reliability index, $\beta_{service}$, can be determined as follows (Ditlevsen and Madsen 1996):

$$\beta_{service} = -\Phi^{-1}(P_{f, service}) \quad (10.11)$$

where Φ is the CDF of the standard normal distribution. $C_{f, top}$, $D_{f, top}$, $C_{f, bottom}$, and $D_{f, bottom}$ need to be modeled such that the $g_{service}$ and then $\beta_{service}$ can be determined.

10.4.2. Probabilistic modeling of stress capacity of extreme fibers at midspan

Table 2-6 provides the limiting compressive and tensile stress criteria per AASHTO (2007). In this research, these limiting values are considered as the stress capacity (C_f) at extreme fibers. For a simply supported beam, the C_f for the top fiber, $C_{f, top}$, is the compressive stress limit and the C_f for the bottom fiber, $C_{f, bottom}$, is the tensile stress limit. These limits are prescribed in AASHTO (2007) and are based on the specified compressive strength of concrete, f'_c . However, because the actual compressive strength of concrete, f_c , is a random parameter, the f'_c needs to be replaced with f_c . A reasonable probabilistic expression for f_c is provided in Subsection 10.5.4. Based on these modified stress limits, the expressions for $C_{f, top}$ and $C_{f, bottom}$ are mathematically expressed as follows:

$$\begin{aligned} C_{f, top} \text{ (ksi)} &= 0.45\sqrt{f_c \text{ (in ksi)}} \\ C_{f, bottom} \text{ (ksi)} &= 0.0948\sqrt{f_c \text{ (in ksi)}} \end{aligned} \quad (10.12)$$

10.4.3. Probabilistic modeling of stress demand on extreme fibers at mid-span

Subsection 2.7.6 reviewed the formulations to determine the applied compressive and tensile stresses (i.e., stress demands) at service at the extreme fibers. Estimating the stresses at extreme fibers is straightforward when the strands are in “as-received” condition. However, the C_T of strands reduces as a function of time and exposure conditions, resulting in changes in these stresses. Typically, D_M is in the linear region of the moment–curvature diagram. However, because D_M is a random parameter, some realizations might be large enough that some strands can break. The effective stress, f_{pe} , earlier imposed by these broken strands should be removed from further calculations. Considering these factors, an iterative algorithm to determine the probabilistic $D_{f, top}$ and $D_{f, bottom}$ has been developed. This algorithm is as follows:

1. Determine D_M using the procedures in Subsection 10.3.3. Determine C_M using the procedures in Subsection 10.3.2. If D_M is greater than C_M , then go to Step 8. If D_M is less than or equal to C_M , then go to step 2.
2. Set $\phi_{min} = 0$, $\phi_{max} = 0.01$, and $\delta_\phi = 1 \times 10^{-6}$ radians/inch.

3. Set the curvature, $\phi = (\phi_{\min} + \phi_{\max})/2$. [Note: These points are moved to either leftward or rightward based on a decision process as described later in step 6.]
4. Call the subfunction to calculate bending moment (This subfunction is presented in 10.3.2.2). Calculate bending moment corresponding to ϕ (i.e., M_ϕ). Also, obtain the updated values of effective prestress force for each strand, $P_{e,j}$, which will be equal to 0 for completely corroded or broken strands and equal to $A_{ps,j} \times f_{pe,j}$ for uncorroded, partially corroded, and unbroken strands.
5. Determine $\delta_M = ||D_M| - |M_\phi||$.
6. If $M_\phi > 0$ and $\delta_M > 0.01 \times |D_M|$ and $M_\phi < D_M$, then set $\phi_{\min} = \phi$ to move the ϕ rightward and repeat steps 3 through 5. If $M_\phi > 0$ and $\delta_M > 0.01 \times |D_M|$ and $M_\phi > D_M$, then set $\phi_{\max} = \phi$ to move the ϕ leftward and repeat steps 3 through 5. [Note: Possible positions of ϕ in the second iteration (i.e., after moving either leftward or rightward) are shown in Figure 10-7. Also, note that Figure 10-7 does not show these positions in further iterative processes or until a solution is obtained.] If $\delta_M \leq 0.01 \times |D_M|$, then go to step 7.
7. Determine the $D_{f, top}$ and $D_{f, bottom}$ using Equation (10.13) (PCI 1999, Nawy 2003, and Naaman 2004):

$$\begin{aligned}
 D_{f, top} &= \sum_{j=1}^{N_{strands}} \left\{ -\frac{P_{e,j}}{A_c} \left(1 - \frac{e_j c_t}{r^2} \right) \right\} - \left(\frac{D_M}{S^t} \right) \\
 D_{f, bottom} &= \sum_{j=1}^{N_{strands}} \left\{ -\frac{P_{e,j}}{A_c} \left(1 + \frac{e_j c_b}{r^2} \right) \right\} + \left(\frac{D_M}{S_b} \right)
 \end{aligned} \tag{10.13}$$

where P_e is the effective prestress force after losses; e is the eccentricity of each strand from the center of gravity of the concrete cross-section; r is the radius of gyration; A_c is the area of the concrete cross-section; S^t and S_b are section moduli of concrete section about the extreme top and bottom fibers, respectively; and c_t and c_b are the distance of the extreme top and bottom fibers, respectively, from the centroid of the concrete cross-section. Go to step 9.

8. Set $D_{f, top} = (1 + C_{f, top})$ and $D_{f, bottom} = (-1 + C_{f, bottom})$. This is done such that the limit state function will become -1 . Note that tensile force is negative and compressive force is positive.
9. Stop.

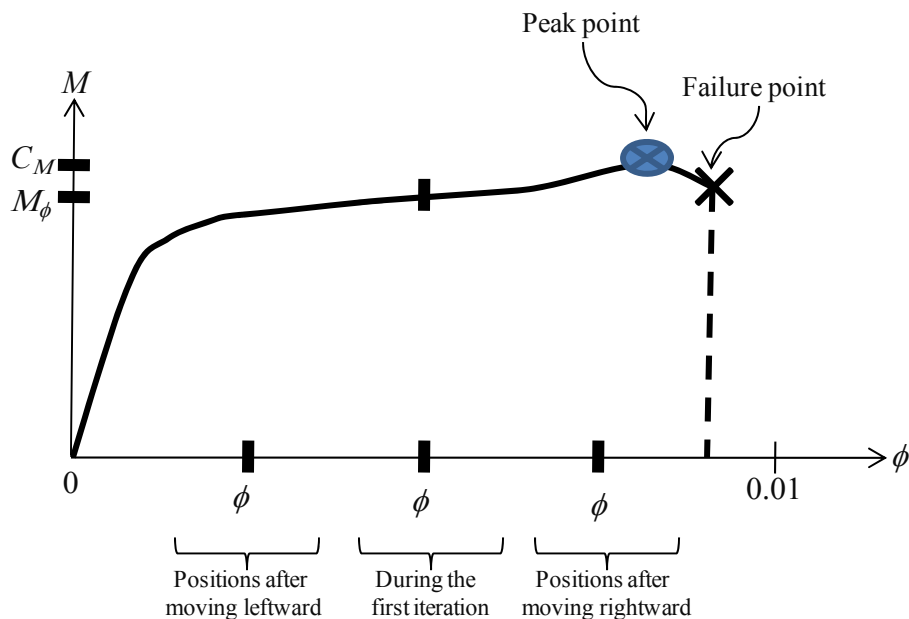


Figure 10-7. Schematic Showing the Movement of Curvature, ϕ .

A model to determine $D_{f, top}$ and $D_{f, bottom}$ of a PT bridge with unbonded tendons that are exposed to various corrosive conditions is now developed.

10.5. RANDOM PARAMETERS IN THE PROBABILISTIC MODELS

Subsections 10.3 and 10.4 developed the strength and service reliability models, respectively. This subsection presents the random parameters used in the probabilistic models in the strength and service reliability analysis.

10.5.1. Void and damage/opening conditions on post-tensioning systems

In this study, void condition (VC) and damage condition (DC) are defined on the basis of the probability of finding a voided tendon (P_{VT}) and the probability of finding a damaged tendon (P_{DT}). In this study, a tendon with at least one void is considered a voided tendon. A damaged tendon is defined as a tendon with at least one unsealed hole or vent at the anchorage region or an unsealed vent or crack on HDPE ducts. As Woodward (1981) and Woodward et al. (2001) determined, P_{VT} can be determined based on inspection of sample bridges or tendons or assumed

based on sound engineering judgment. Similarly, P_{DT} can be determined or assumed. Based on P_{VT} and P_{DT} , VC and DC are modeled as binomial distributions. Binomial distribution is suitable to randomly identify the tendons with and without voids and with and without damages. Based on the data from TxDOT (2004), P_{VT} and P_{DT} are assumed to be 78.6 and 12 percent, respectively, in this study. These can be statistically expressed as follows:

$$\begin{aligned} VC &\sim B(1, 0.786) \\ VD &\sim B(1, 0.12) \end{aligned} \tag{10.14}$$

10.5.2. Tension capacity of strands

Sections 7 and 5 developed probabilistic models for the C_T of strands under various exposure conditions. Four of these models are used to assess the structural reliability based on a model selection strategy provided in Figure 10-8. These models are summarized here. Table 10-1 and Table 10-2 show definitions of the terms and the median estimates of the unknown model parameters, Θ , as defined in Subsection 7.2.2 and 7.2.3. The standard deviations and correlation coefficients of these parameters are shown in Sections 7 and 9.

$$\begin{aligned} \text{Model 1: } C_T &\sim \text{LN}(59.27, 0.29) \text{ kips} \\ &\sim \text{LN}(263.7, 1.3) \text{ kN} \end{aligned} \tag{10.15}$$

$$\begin{aligned} \text{Model 2: } C_T &= C_{T,n} \left[A \left(\gamma_{t_{CA}} \right)^{n_{CA}} + \sigma \varepsilon \right] \\ \text{where, } A &= \theta_1 \left\{ \theta_2 - \theta_3 \exp(\gamma_{RH}) - \theta_4 \exp \left[\gamma_{\%g_{CF}} \exp(\gamma_{RH}) \gamma_T \right] \right\}^{\theta_1} \end{aligned} \tag{10.16}$$

$$\text{Model 3: } C_T = C_{T,n} \left[\theta_1 \left[\theta_2 - \theta_3 \gamma_{\%s_{CF}} \gamma_{t_{WD}} \right]^{\theta_4} + \sigma \varepsilon \right] \tag{10.17}$$

$$\text{Model 4: } C_T = C_{T,n} \left[\theta_1 \left[\theta_2 - \theta_3 \gamma_{t_{WD}} - \theta_4 \ln[\gamma_{\%sCl^-}] \gamma_{t_{WD}} \right]^{\theta_5} + \sigma \varepsilon \right] \quad (10.18)$$

Table 10-1. Definitions of the Terms in the Strand Capacity Models.

Terms	Definitions
$C_{T,n}$	Nominal tension capacity of "as-received" strand
θ_i	Unknown model parameter
σ	Standard deviation of model error
ε	Standard normal variable $\sim N(0,1)$
$\gamma_{t_{CA}}$	$\frac{\text{Total atmospheric exposure time (years)}}{\text{standardizing factor}} = \frac{t_{CA}}{0.75}$
n_{CA}	Constant term based on field information
γ_T	Ambient exposure temperature ($^{\circ}\text{F}$) = $T(^{\circ}\text{F})$
γ_{RH}	$\frac{\text{Ambient relative humidity (\%)}}{\text{Maximum Relative humidity (\%)}} = \frac{RH (\%)}{100}$
$\gamma_{\%gCl^-}$	$\frac{\%Cl^- \text{ in the grout (by weight)}}{\%sCl^-_{\text{saturated chloride solution}}} = \frac{\%gCl^-}{35.7}$
$\gamma_{\%sCl^-}$	$\frac{\%Cl^- \text{ in the water inside the tendon}}{\%sCl^-_{\text{saturated chloride solution}}} = \frac{\%sCl^-}{35.7}$
ϕ_{wet}	$\left(\frac{\text{Wet-time in a year (months)}}{12 \text{ (months)}} \right)$; $0 \leq \phi_{wet} \leq 1$
$\gamma_{t_{WD}}$	$\phi_{wet} \times \text{Total exposure time (years)} = \phi_{wet} \times t_{WD}$

Table 10-2. Posterior Statistics of Model Parameters in the Strand Capacity Models.

	Model 2	Model 3	Model 4
θ_1	7.7492	0.9983	0.9463
θ_2	0.1637	1.0105	1.0333
θ_3	0.0030	1.6785	0.3567
θ_4	0.0002	1.3576	0.0285
θ_5	1.0924	-	2.0301
σ	-	0.0117	0.0350

For each Monte Carlo simulation, an appropriate probabilistic model for C_T of each strand is selected based on the tendon type, environmental condition (EC), and randomly selected VC and DC . Figure 10-8 shows the strategy used to select the strand capacity models,

where the circles indicate the model numbers for each exposure category (i.e., combinations of tendon type, *EC*, *VC*, and *DC*). The *EC* is selected manually (per Figure 10-10) and *VC* and *DC* are selected randomly (per Equation (10.14)). The two *VC*s considered are “No void” and “Void” conditions. The two *DC*s considered are “No damage” and “Damage” conditions. Because the “Wet-Dry” condition occurs only if there are damages on the tendon system, the “No damage” condition corresponding to the “Wet-Dry” conditions is not shown in Figure 10-8.

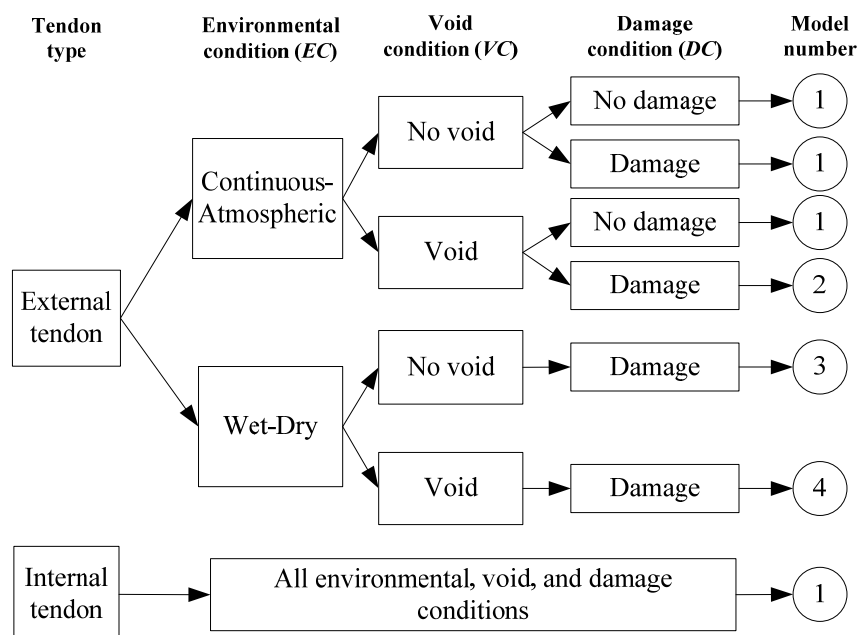


Figure 10-8. Strategy to Select the Strand Capacity Models Based on Exposure Conditions.

10.5.3. Prestress loss of strands

The literature review provided in Subsection 2.7.1.2 did not identify sufficient information to develop a probabilistic expression for P_{loss} for PT bridge tendons. AASHTO (2007) provides a lump sum estimate of 19 ksi (131 MPa) for prestress losses in box girders. There exist some cautions (see Subsection 2.7.1.2) in using this lump sum loss estimate for deterministic structural assessment of segmental bridges. P_{loss} corresponding to 19 ksi (131 MPa) is 4123 lbs (18 kN)

for a 0.6-inch (15 mm) diameter strand. In this study, P_{loss} is assumed to follow a lognormal distribution with a mean value equal to this lump sum estimate. Because the COV of P_{loss} could not be estimated from field data, in this study it is assumed to be 0.15. P_{loss} can be statistically expressed as follows:

$$\begin{aligned} P_{loss} &\sim \text{LN}(4123, 618) \text{ lbs} \\ &\sim \text{LN}(18, 2.8) \text{ kN} \end{aligned} \quad (10.19)$$

10.5.4. Compressive strength of concrete

According to TxDOT (2004), Class H concrete with f'_c equal to 6 ksi (41.3 MPa) was used in the construction of precast segments in Texas bridges constructed in the 1980s and 1990s. Because of liability reasons, in general, contractors provide concrete with f_c that is higher than f'_c . Hence, a mean compressive strength, $\overline{f_c}$, of 6 ksi (41.3 MPa) would be reasonable for this study. Based on the information in Table 2-5, assuming a COV of 0.15 for concrete with f'_c equal to 6 ksi would be reasonable. This will result in a calculated standard deviation of 0.7 ksi. Because f_c is always a positive number, it is expressed as a lognormal distribution as follows:

$$\begin{aligned} f_c &\sim \text{LN}(6, 0.9) \text{ ksi} \\ &\sim \text{LN}(41.34, 6.2) \text{ MPa} \end{aligned} \quad (10.20)$$

10.5.5. Dead and live load parameters

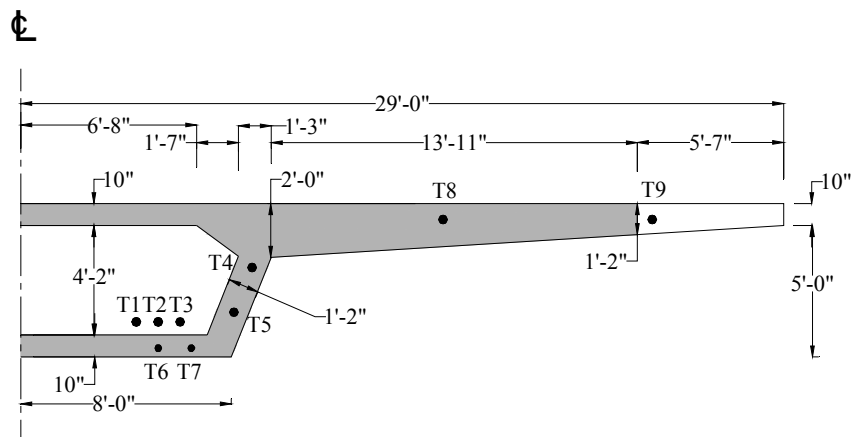
The dead load due to concrete box girder (DL_1) and dead load due to concrete overlay, future concrete overlay, and concrete side barriers (DL_2) are always positive and, therefore, are assumed to follow lognormal distributions. The mean value of these parameters can be obtained using the cross-sectional geometry (see Subsection 10.6.1) and unit weight of the materials (see Table 2-7) of the selected PT girder. The standard deviation can be obtained by assuming a COV equal to 0.10 (see Subsection 2.8.12.8.2).

The live load parameters, LL_{lane} , LL_{truck} , and LL_{tandem} , are assumed to follow normal distributions. The mean values are obtained using the standard procedures provided in AASHTO (1996) for HS20 loading and AASHTO (2007) for HL93 loading conditions. Based on the discussions in Subsection 2.8.2, a COV of 0.18 is assumed after the impact load is appropriately added to the live load combinations. The authors realize that HL93 loading was not in place at the time of construction of some of the PT bridges in Texas. Several references have noted that HL93 loading (with truck and lane loading) is more realistic loading condition than the HS20 loading (truck loading only). Therefore, we also performed the analysis to assess the bridge under a more realistic current loading condition (i.e., HL93 loading).

10.6. TIME-VARIANT STRUCTURAL RELIABILITY ASSESSMENT OF TYPICAL POST-TENSIONED BRIDGE

10.6.1. Geometrical and structural characteristics of a typical post-tensioned bridge

Based on the maximum number of spans used on bridges in Texas, the Type III girder has been selected for the structural reliability assessment. Figure 10-9 shows the cross-section of a typical Type-III girder. Table 2-11 provides the depths of the tendons in this cross-section at midspan. The shaded region in Figure 10-9 indicates the effective cross-section used to estimate $g_{strength}$ and $g_{service}$. The effective flange width, b_e , on each side of the web is calculated using the formulations and empirical curves provided in Article 4.6.2.6.2 of AASHTO (2007), and widths are summarized in Table 10-3. The girder in this study is assumed to be simply supported. The D_M calculations are performed based on this assumption. It is important to note that the selected cross-section is similar to the Type-III girders on the San Antonio “Y” bridge. However, because of the sequence of the post-tensioning and support removal process, the San Antonio “Y” bridge behaves somewhere in between simply supported and continuous beam. Hence, the obtained reliability results do not necessarily represent that of the San Antonio “Y” bridge. Further modifications to the developed models and MATLAB[®] programs are needed to conduct an analysis of a specific bridge.



Notes:

- 1) The cross-section is symmetrical about the center-line.
- 2) Nominal diameter of all the strands is 0.6 inch (15 mm).
- 3) One half of the cross-section contains 9 tendons.
- 4) Tendons T1, T2, T3, T4, T5, T8, and T9 contain 19 strands each.
- 5) Tendons T6 and T7 contains 12 strands each.
- 6) Tendons T8 and T9 are not used in flexural capacity calculations.
- 7) Shaded region indicates effective cross-section, per AASHTO LRFD (2007).

Figure 10-9. Cross-Section at Mid-span of the Typical PT Girder Used in This Study.

Table 10-3. Effective Flange Widths for Each Side of the Web.

	<i>b</i> inch (m)	<i>l_i</i> inch (m)	<i>b/l_i</i>	<i>b_e/b</i>	<i>b_e</i> inch (m)
b1	234 (5.94)	1200 (30.48)	0.195	0.74	173 (4.40)
(b3 + b4)	99 (2.51)	1200 (30.48)	0.083	0.95	94 (2.39)
(b5 - t_w)	82 (2.08)	1200 (30.48)	0.068	0.97	80 (2.02)

b1, b3, b4, b5, and t_w are defined in Figure 2-22 and Table 2-12. *b*, *b_e*, and *l_i* are physical width, effective width, and notional span, specified in Article 4.6.2.6.2 of AASHTO (2007).

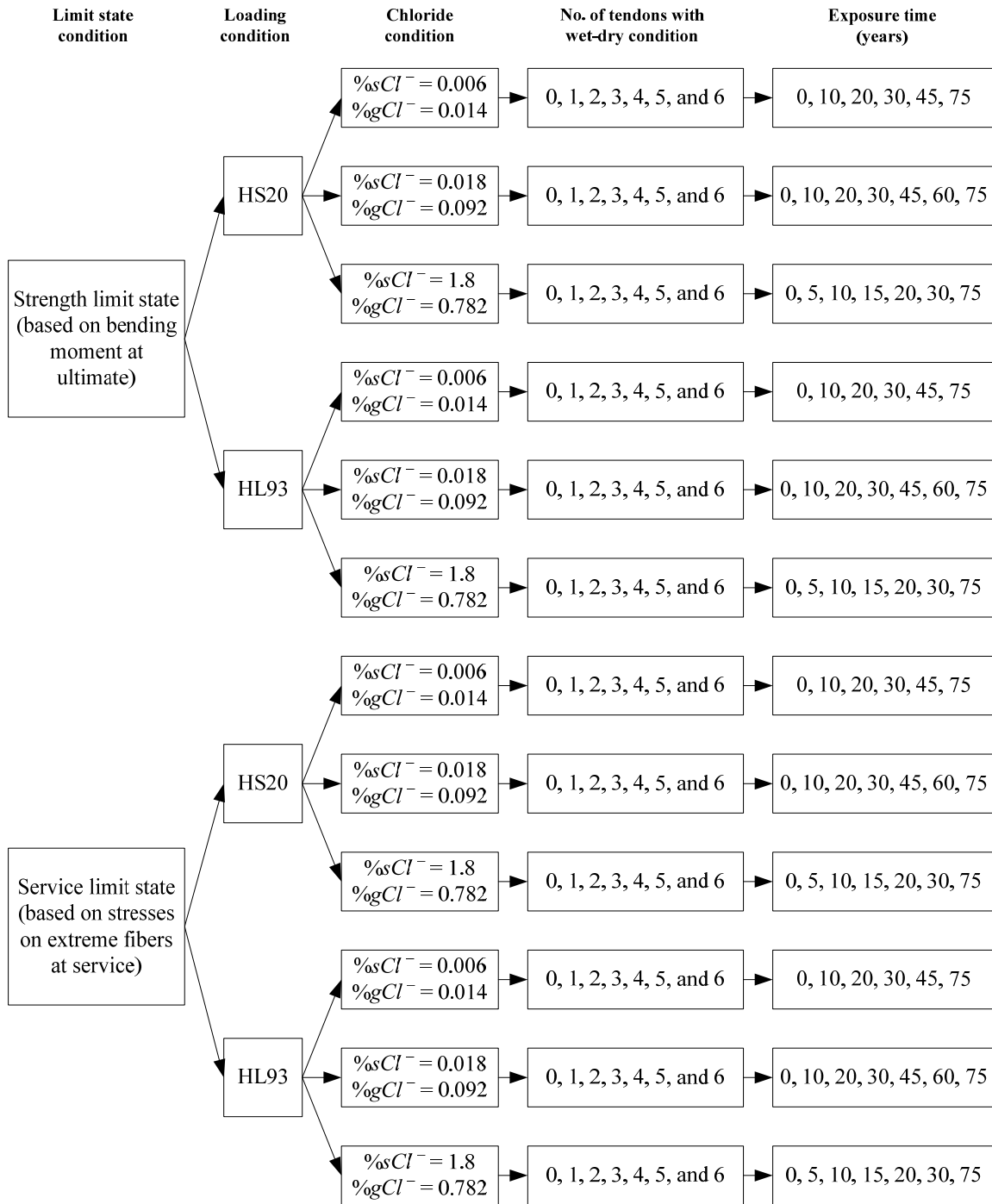
10.6.2. Definitions of parameters for the reliability assessment

In this study, $\beta_{strength}$ and $\beta_{service}$ of the typical PT bridge experiencing HS20 and HL93 loading conditions will be assessed. In the current research, $COV(P_f)_{target}$ has been defined as 5 percent. However, there are some cases where this criterion could not be met due to the limited computing resources. The actual $COV(P_f)$ s achieved are reported in APPENDIX C.

$\beta_{strength}$ and $\beta_{service}$ of the typical PT bridge with six external tendons for various combinations of loading and exposure conditions will be assessed using the developed strength and service reliability models. These parameter combinations are shown in Figure 10-10. The exposure times in Figure 10-10 are selected based on the expected time for complete corrosion of strands under certain exposure conditions. For example, when exposed to 0.006, 0.018, and 1.8 %s Cl^- conditions (with ϕ_{wet} equal to 0.17), the time required for the strands to completely corrode are 57, 45, and 23 years, respectively. The time intervals for each %s Cl^- condition were selected such that maximum information is obtained to effectively capture the change in β .

Table 10-4 shows that the average annual relative humidity and temperature of most cities in the United States are approximately 70 percent and 70°F, respectively. Based on this, the values of T and RH are assumed to be 70°F (21.1°C) and 70 percent, respectively, for the reliability analysis. Also, the values of ϕ_{wet} and n_{CA} are assumed to be 0.17 and -0.005, respectively. The effects of WD conditions with 0.006, 0.018, and 1.8 %s Cl^- levels in 0, 1, 2, 3, 4, 5, and 6 external tendons on $\beta_{strength}$ and $\beta_{service}$ will be assessed. The tendons without WD conditions will be assumed to have a CA condition with 0.014, 0.092, or 0.782 %g Cl^- level. The exposure periods at which $\beta_{strength}$ and $\beta_{service}$ will be determined are provided in the fifth column in Figure 10-10.

Table 10-5 summarizes all the random parameters used in this study. Note that the dead and live load parameters (i.e., DL and LL) in Table 10-5 are calculated based on the geometry of the typical PT bridge being studied.



Other parameters:

Ambient $RH = 70\%$; RH inside undamaged tendons = 50% ;

$T = 70^\circ F (24.4^\circ C)$; $\phi_{wet} = 2/12 = 0.17$; $n_{CA} = -0.005$

Figure 10-10. Parameter Combinations for the Reliability Assessment Program.

Table 10-4. Average Annual Temperature and Relative Humidity Conditions in Some Major Cities in the US (Source: www.cityrating.com).

Name of the US city	Average annual ambient temperature, T (°F)	Average annual ambient relative humidity, RH (%)
Atlanta	61	69
Austin	69	71
Chicago	49	71
Dallas-Fort Worth	65	70
Houston	68	77
Los Angeles	63	72
Miami	76	72
New York	55	64
Orlando	72	73
San Antonio	69	70
Seattle	53	73
Tampa	72	74
Washington, D.C.	54	69

Table 10-5. Random Parameters for Reliability Assessment*.

Parameter	Distribution type	Statistics
VC	Binomial	$\sim B(1, 0.786)$
DC	Binomial	$\sim B(1, 0.12)$
σ	Normal	$\sim N(0, 1)$
P_{loss}	Lognormal	$\sim LN(4123, 618)$ kips
f_c	Lognormal	$\sim LN(6, 0.9)$ ksi
DL_1	Lognormal	$\sim LN(2.69, 0.67)$ kip/ft
DL_2	Lognormal	$\sim LN(14.31, 1.43)$ kip/ft
LL_{lane}	Normal	$\sim N(0.512, 0.092)$ kip/ft
LL_{truck}	Normal	$\sim N(25.6, 4.6)$ kips
LL_{tandem}	Normal	$\sim N(20.0, 3.6)$ kips

* 1 kip = 4.45 kN; 1 ksi = 6.89 kN/mm²; 1 kip/ft = 14.6 kN/m.

10.6.3. Time-variant strength reliability index

The time-variant $\beta_{strength}$ and $P_{f, strength}$ of the selected PT bridge was assessed using the strength reliability model developed in Subsection 10.3. The parameter combinations for this parametric study were presented in Figure 10-10. The ISO 13822 (2001) standard suggests β_{target} values of 2.3, 3.1, 3.8, and 4.3 for structures with very low, low, medium, and high consequences of failure, respectively (see Table 2-9). The consequence of failure of massive amounts of structures in a city due to an earthquake could be considered medium or high. The consequence of failure of a nuclear power plant could also be considered medium or high (depending on the toxicity of radiation and the proximity to a human populated area). In comparison to these cases, the consequence of failure of a typical segmental bridge may be considered low, especially for segmental, PT bridges on urban highways. The following discussions include comparisons of the estimated reliability with β_{target} used in AASHTO (2007) code calibration (i.e., 3.5) and recommended by ISO 13822 (2001) for strength failure with low consequences of failure (i.e., 3.1). Note that in this analysis, all the internal tendons were assumed to be intact and protected from corrosion, which might not be the actual case in the field.

Figure 10-11, Figure 10-12, and Figure 10-13 show the variation of $\beta_{strength}$ and $P_{f, strength}$ for up to 75 years of exposure. Note that the ϕ_{wet} used in this analysis is equal to 0.17, which corresponds to 2 months of wet time in every year. The horizontal lines with long dashes in these figures indicate the basis for the calibration of AASHTO (2007) document for ultimate conditions. The horizontal lines with short dashes in these figures indicate the β_{target} (i.e., 3.1) for a failure case with low consequences of failure (per ISO 13822 [2001]). The dashed and solid curves indicate the cases with HS20 and HL93 loading conditions, respectively. The dashed and solid data markers also indicate the cases with HS20 and HL93 loading conditions, respectively. Different data markers are used to represent the number of tendons subjected to WD exposure cycles, defined as “WDT.” If $\beta_{strength}$ of a particular case is above 3.5 and could not be obtained due to the limited computing resources and time, the corresponding curves are not shown.

For the selected PT bridge, it seems that the $\beta_{strength}$ remained above 3.5 when all the strands were in “as-received” condition. However, the actual value of $\beta_{strength}$ for this case could not be determined because the number of failure cases after 10 million Monte Carlo simulations

was 0 and the simulation process was ended due to limited computing resources and time required to attain a $COV(P_f)_{target}$ of 0.05. In addition, no failure was observed among 10 million simulations when only one tendon (i.e., 19 strands), which constitutes about 8 percent of total tendons, was completely corroded or failed.

10.6.3.1. Strength reliability when exposed to 0.006 %sCl⁻ conditions

Figure 10-11 shows the variation of $\beta_{strength}$ with time when exposed to 0.006 %sCl⁻ solution. When only two tendons are subjected to HS20 loading and exposed to 0.006 %sCl⁻ solution (hollow square markers), the value of $\beta_{strength}$ at 75 years is above 3.5. For this case, when subjected to HL93 loading instead of HS20 loading (solid square markers), the $\beta_{strength}$ drops below 3.5 in about 30 years and stays above 3.1 up to 75 years. When more than two tendons are exposed to WD cycles (all triangular markers), the maximum time needed for $\beta_{strength}$ to drop below 3.5 is 25 years and to drop below 3.1 is 35 years. In the most severe case with HL93 loading and 6 WDTs, it takes only about 12 years for $\beta_{strength}$ to drop below 3.1.

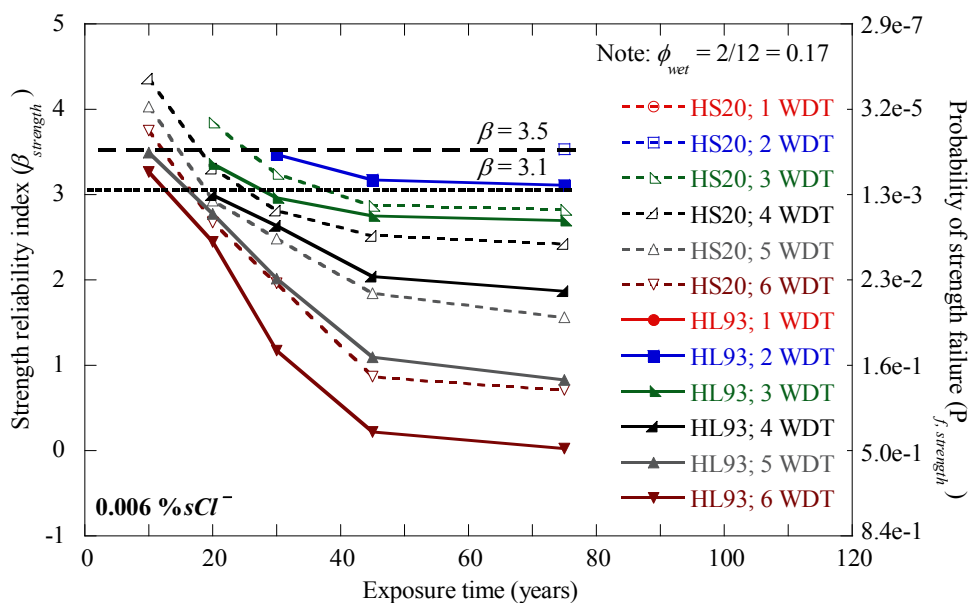


Figure 10-11. Strength Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (0.006 %sCl⁻).

10.6.3.2. Strength reliability when exposed to 0.018 %sCl⁻ conditions

Figure 10-12 shows the variation of $\beta_{strength}$ with time when exposed to 0.018 %sCl⁻ solution. If only two tendons are subjected to HS20 loading and exposed to 0.018 %sCl⁻ solution (hollow square markers), then the value of $\beta_{strength}$ stays above 3.5. For this case, when subjected to HL93 loading instead of HS20 loading (solid square markers), the $\beta_{strength}$ drops below 3.5 in about 22 years and stays above 3.1 up to 75 years. When three or more tendons are exposed to WD cycles (all triangular markers), the maximum time needed for $\beta_{strength}$ to drop below 3.1 is approximately 28 years, as opposed to 35 years in the case with exposure to 0.006 %sCl⁻ solution. In the most severe case with HL93 loading and 6 WDTs, it seems to take only about 8 years for $\beta_{strength}$ to drop below 3.1, as opposed to 12 years in the case with exposure to 0.006 %sCl⁻ solution.

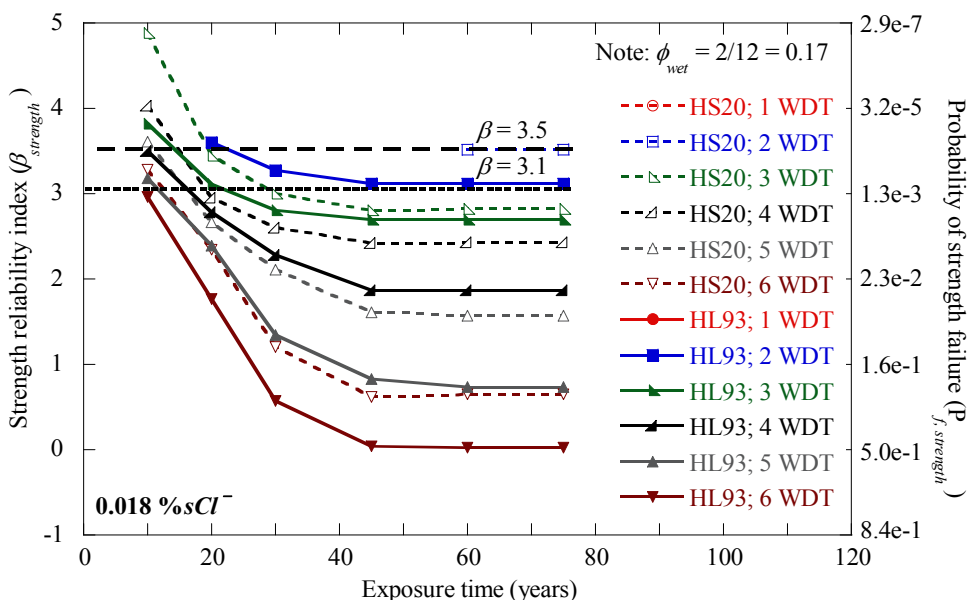


Figure 10-12. Strength Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (0.018 %sCl⁻).

10.6.3.3. Strength reliability when exposed to 1.8 %sCl⁻ conditions

Figure 10-13 shows the variation of $\beta_{strength}$ with time when exposed to 1.8 %sCl⁻ solution. When the bridge is subjected to HS20 or HL93 loading and there is only one WDT (round markers), $\beta_{strength}$ stays above 3.5. However, because exact $\beta_{strength}$ values for these cases could not be obtained, they are not shown in Figure 10-13. When the bridge is subjected to HS20 or HL93 loading and there are two WDTs (square markers), $\beta_{strength}$ drops below 3.5 and stays above 3.1. When three or more tendons are exposed to WD cycles (all triangular markers), the maximum time needed for $\beta_{strength}$ to drop below 3.1 is approximately 13 years, as opposed to 35 and 28 years in the cases with exposure to 0.006 and 0.018 %sCl⁻ solutions, respectively. In the most severe case with HL93 loading and 6 WDTs, it seems to take less than 5 years for $\beta_{strength}$ to drop below 3.1, as opposed to 12 and 8 years, respectively, in the cases with exposure to 0.006 and 0.018 %sCl⁻ solutions.

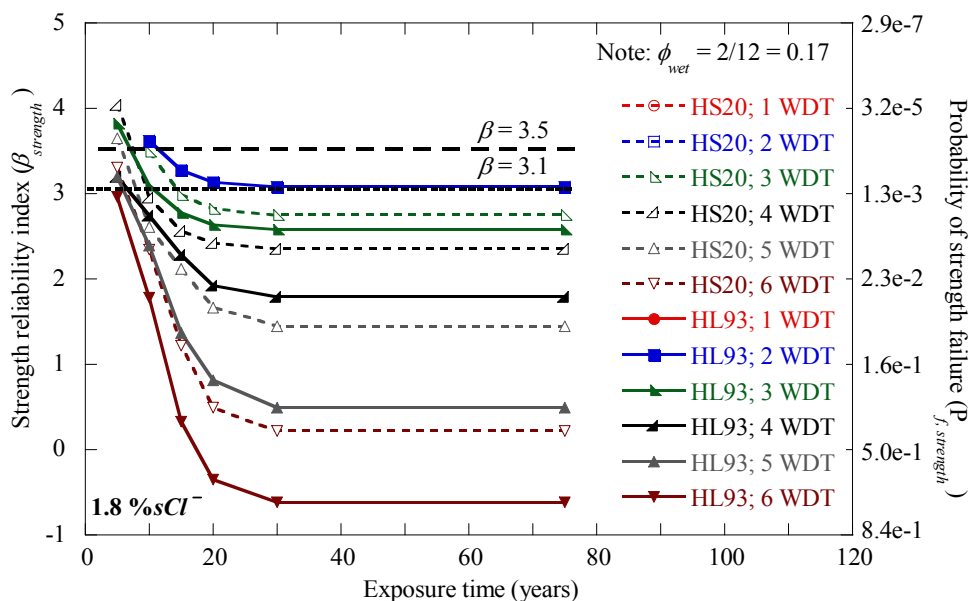


Figure 10-13. Strength Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (1.8 %sCl⁻).

10.6.4. Time-variant service reliability index

The time-variant $\beta_{service}$ and $P_{f, service}$ of the selected PT bridge was assessed using the service reliability model developed in Subsection 10.4. The parameter combinations for this study were the same as those used for the strength reliability assessment (see Figure 10-10). The ISO 13822 (2001) standard suggests β_{target} values of 0 and 1.5 for reversible and irreversible consequences of failure, respectively (see Table 2-9). There are concrete materials that are capable of self-healing microcracks. When the tensile stresses exceed the limit state, cracks may form and may not self-heal, depending on the type of concrete material used. Such cases could be defined to have an irreversible consequence of failure. In addition, exceeding the compressive stress limits can induce changes in concrete's material properties and could be considered to have an irreversible consequence of failure. However, there may also be cases with reversible consequences of failure (such as in the case of concrete with good self-healing of tensile cracks and elastic behavior in compression). The following discussions include comparisons of the estimated service reliability with β_{target} recommended by ISO 13822 (2001) for service failures with reversible and irreversible consequences of failure (i.e., 0 and 1.5).

Figure 10-14, Figure 10-15, and Figure 10-16 show the variation of $\beta_{service}$ and $P_{f, service}$ for up to 75 years of exposure. The ϕ_{wet} used in this analysis is equal to 0.17, which corresponds to 2 months of wet time in every year. The horizontal lines with short and long dashes in these figures indicate the β_{target} values for reversible and irreversible consequences of service failure (per ISO13822 [2001]). The dashed and solid curves indicate the cases with HS20 and HL93 loading conditions, respectively. The dashed and solid data markers also indicate the cases with HS20 and HL93 loading conditions, respectively. Different data markers are used to represent the number of tendons subjected to WD exposure cycles, defined as "WDT."

For the selected PT bridge, $\beta_{service}$ is above 1.5 (i.e., the recommended value for the case with irreversible consequences of failure) if the structure experiences HS20 loading and strands are in "as-received" condition. However, $\beta_{service}$ is below 1.5 and above 0 (i.e., the recommended value for the case with reversible consequences of failure) if the structure experiences HL93 loading and strands are in "as-received" condition. The following is a presentation on the effect of various exposure conditions and strand corrosion on $\beta_{service}$.

10.6.4.1. Service reliability when exposed to 0.006 %sCl⁻ conditions

Figure 10-14 shows the variation of $\beta_{service}$ with time when exposed to 0.006 %sCl⁻ solution. When there is only one WDT and the bridge is subjected to HS20 loading (hollow round markers), the value of $\beta_{service}$ stays above 1.5 for up to 22 years. For this case, when the bridge is subjected to HL93 loading (solid round markers), the value of $\beta_{service}$ stays above 0 for up to 53 years. When there are only two WDTs (square markers), these two time estimates (i.e., 22 and 53 years) reduce to about 10 and 16 years, respectively. If the bridge is subjected to HS20 loading and there are three or more WDTs (all triangular markers), then the maximum time needed for $\beta_{service}$ to drop below 1.5 and 0 is about 7 and 35 years, respectively. For this case, the minimum time needed for $\beta_{service}$ to drop below 1.5 and 0 is about 2 and 13 years, respectively.

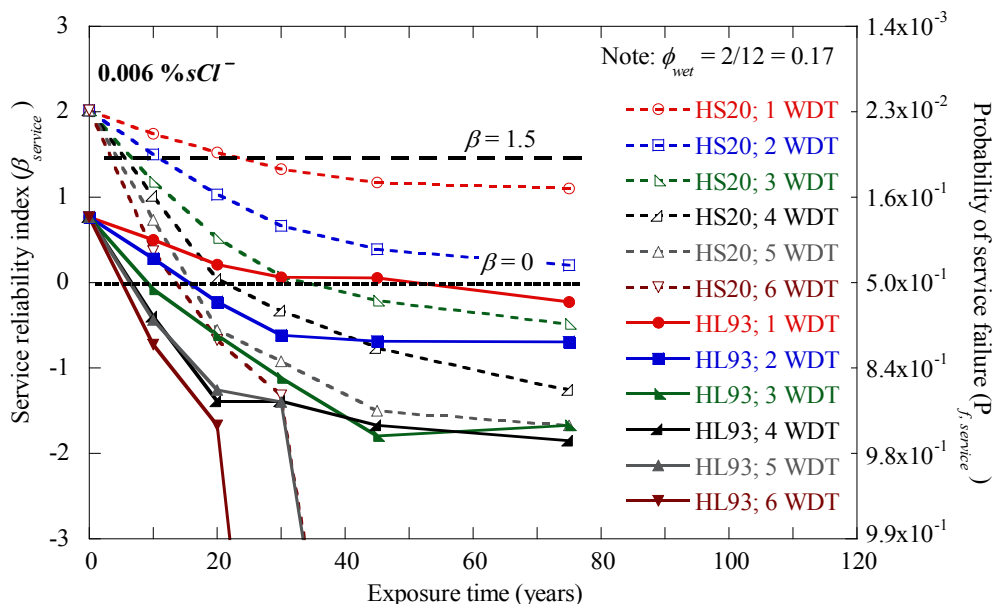


Figure 10-14. Service Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (0.006 %sCl⁻).

10.6.4.2. Service reliability when exposed to 0.018 %sCl⁻ conditions

Figure 10-15 shows the variation of $\beta_{service}$ with time when exposed to 0.018 %sCl⁻ solution. When there is only one WDT and the bridge is subjected to HS20 loading (hollow round markers), the value of $\beta_{service}$ stays above 1.5 for up to 18 years, as opposed to 22 years in the case of exposure to 0.006 %sCl⁻ solution. For this case, when the bridge is subjected to HL93 loading (solid round markers), the value of $\beta_{service}$ stays above 0 for up to about 45 years, as opposed to about 53 years in the case of exposure to 0.006 %sCl⁻ solution. When there are only two WDTs (square markers), these two time estimates (i.e., 18 and 45 years) significantly reduce to about 9 and 25 years, respectively. If the bridge is subjected to HS20 loading and there are three or more WDTs (all triangular markers), then the maximum time needed for $\beta_{service}$ to drop below 1.5 and 0 is 5 and 25 years, respectively. For this case, the minimum time needed for $\beta_{service}$ to drop below 1.5 and 0 is about 2 and 10 years, respectively.

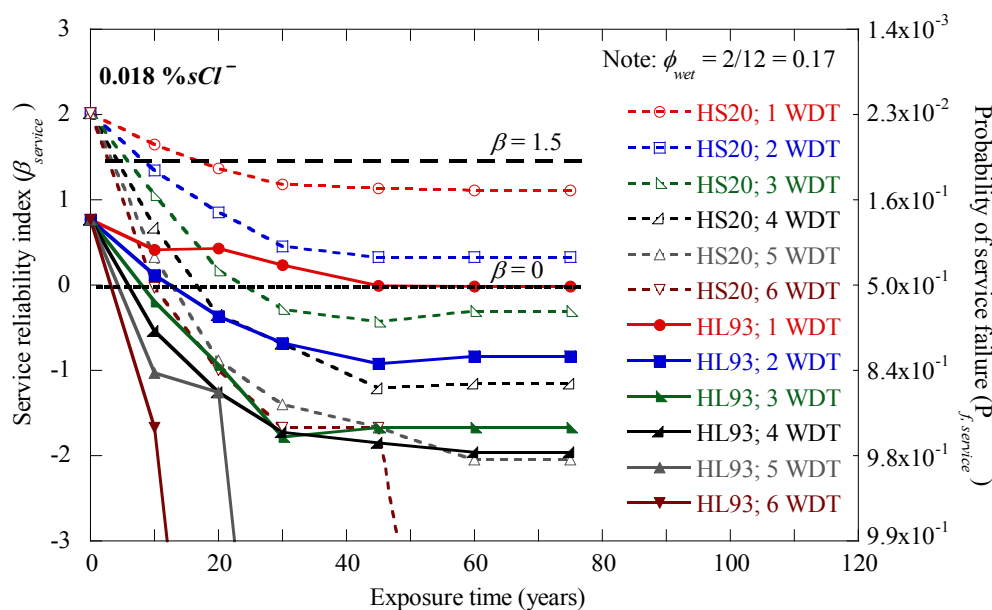


Figure 10-15. Service Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (0.018 %sCl⁻).

10.6.4.3. Strength reliability when exposed to 1.8 %sCl⁻ conditions

Figure 10-16 shows the variation of $\beta_{service}$ with time when exposed to 1.8 %sCl⁻ solution. When there is only one WDT and the bridge is subjected to HS20 loading (hollow round markers), the value of $\beta_{service}$ stays above 1.5 for up to 10 years, as opposed to 22 and 18 years in the cases of exposure to 0.006 and 0.018 %sCl⁻ solutions. For this case, HL93 loading exists (solid round markers) and the value of $\beta_{service}$ stays above 0 for up to about 10 years, as opposed to about 53 and 45 years in the cases of exposure to 0.006 and 0.018 %sCl⁻ solutions. When there are only two WDTs (square markers), these two equal time estimates (i.e., 10 and 10 years) significantly reduce to about 3 and 7 years, respectively. If the bridge is subjected to HS20 loading and there are three or more WDTs (all triangular markers), then the maximum time needed for $\beta_{service}$ to drop below 1.5 and 0 is 2 and 11 years, respectively. For this case, the minimum time needed for $\beta_{service}$ to drop below 1.5 and 0 is about 1 and 5 years, respectively.

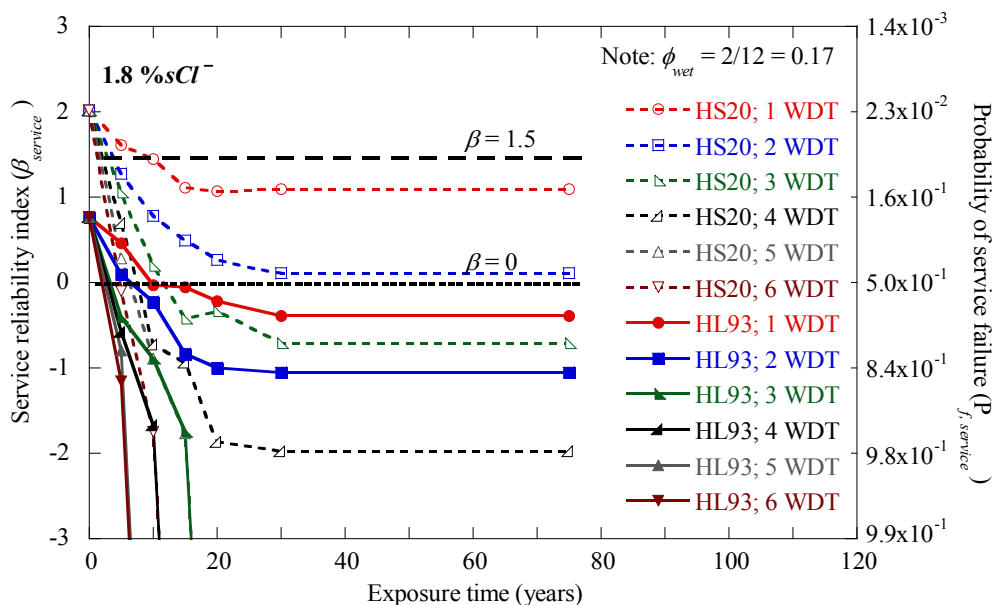


Figure 10-16. Service Reliability of PT Bridges with Tendons Exposed to Wet-Dry Cycles (1.8 %sCl⁻).

10.7. SUMMARY

This section assessed the strength and service reliability of a typical PT bridge. At first, the strength and service reliability models along with the required capacity and demand models were presented. The $\beta_{strength}$ and $\beta_{service}$ of a typical simply supported PT bridge were then assessed using these models. The uncertainty was captured using the probabilistic models for C_T of strands (developed in Section 7) and other random parameters defining the material characteristics of concrete, prestress losses, voids and damage conditions in PT systems, and structural loading conditions. The values of $\beta_{strength}$ and $\beta_{service}$ were determined for up to 75 years for a range of specified environmental conditions and randomly selected void and damage conditions.

11. CONCLUSIONS AND RESEARCH RECOMMENDATIONS

11.1. INTRODUCTION

This section provides conclusions drawn from a comprehensive research project on the effects of void, environmental, and other exposure conditions on the long-term performance of post-tensioned (PT) bridges. At first, environmental characterization maps of Texas were developed. The electrochemical and tension capacity (C_T) behavior of PT systems were then experimentally investigated. Based on these experimental results and engineering judgments and assumptions, probabilistic models to determine C_T of strands were then developed. Following this, time-variant structural reliability models were developed. Using these reliability models, the strength and service reliabilities of a typical PT bridge for a period of 75 years were assessed.

Volume 2 of this report discusses the research on the procedures to inspect voids and damages in PT systems, characteristics of PT grout materials, procedures to repair the voids in PT tendons, and economic risk of PT bridges.

11.2. LIMITATIONS AND ASSUMPTIONS

Limitations and assumptions associated with the results obtained from this research include:

- Sufficient field data could not be obtained to more accurately calibrate the probabilistic models for C_T of strands exposed to WD and CA exposure conditions.
- Engineering judgments and assumptions were made to calibrate the probabilistic models to potential field conditions.
- Probabilistic models for the C_T of strands are used in the structural reliability analysis. Hence, the limitations of the probabilistic models for the C_T of strands are also applicable to the structural reliability assessment.
- All the internal tendons are assumed to be intact and free from corrosion.
- Structural reliability models developed in this research consider only a simply supported condition.

- Structural reliability models developed here consider only uniaxial bending mechanisms. These models do not consider biaxial bending, shear, torsion, or other structural mechanisms, which can influence the system reliability of PT bridges.

Although limitations exist, some valuable conclusions are drawn from this research and are presented next.

11.3. CONCLUSIONS

11.3.1. Corrosion Risks at Different Geographic Locations in Texas

Section 4 presented maps showing freeze-days, temperature, relative humidity, and rain-days in Texas. These maps were developed using Geographic Information Systems (GIS) technology and the data collected from the internet and Texas district engineers. The conclusions drawn from these maps are:

- The majority of Texas has a moderate level of corrosion risk and some areas have a high level of corrosion risk.
- Coastal regions have the most critical conditions due to consistent exposure to chloride-contaminated environments.
- These general maps should be used only for initial screening purposes. Corrosion risk of a specific PT bridge should be assessed based on additional relevant information (such as the presence of voids, damage, moisture, chlorides, etc. inside the tendons).

11.3.2. Cyclic Polarization Curves of ASTM A416 Steel

Subsections 5.3 and 6.2 provided the cyclic polarization test program and the corresponding results, respectively. The conclusions include the following:

- The breakdown potential decreases from approximately 0.5 Volts to 0.4 Volts as the percent chloride concentration in the exposure solution increases from 0 to 1.8. This indicates that the ASTM A416 steel is more susceptible to corrosion when embedded in cementitious materials contaminated with high amounts of chlorides (for example, the strands in grouted tendons infiltrated with seawater).

- The passive region becomes narrower as the chloride level increases. This can likely lead to an increased probability of strand corrosion as chloride levels increase.

11.3.3. Galvanic Corrosion Characteristics of PT Systems

The galvanic corrosion test program and results are provided in Subsections 5.4 and 6.3, respectively. The conclusions include the following:

- The corrosion potentials of conventional reinforcement and prestressing strands are similar, and there was limited galvanic corrosion between conventional reinforcement and prestressing strands.
- The presence of moisture can initiate galvanic corrosion between bearing plates and conventional reinforcement and bearing plates and strands in PT systems. Also, the presence of high amounts of chlorides can accelerate this galvanic corrosion.

11.3.4. Probabilistic Tension Capacity of Strands and Wires

Section 1 presented the experimental program and results on strand and wire corrosion tests. Sections 7, 8, and 9 presented the probabilistic models for C_T of strands and wires. Subsection 5.5.2 presented the definitions and schematics of different void conditions (i.e., NV, PV, OV, IV, BV, and BIOV). The conclusions derived from these results are as follows:

- The corrosion mechanisms for a PT system containing voids and one containing no voids are different, and the presence of the voids has a more significant effect on the corrosion rate than the grout material characteristics, such as chemical composition and water-cementitious materials ratio.
- The moisture level has a statistically significant influence on the corrosion and C_T of PT strands. The C_T of strands exposed to high moisture levels can be up to 17 percent less than the C_T of strands exposed to low moisture levels, provided other exposure conditions remain the same.
- The chloride level has a statistically significant influence on the corrosion rate and C_T of PT strands. The C_T of strands exposed to high chloride levels can be up to 25 percent less than the C_T of strands exposed to negligible chloride levels, provided other exposure conditions remain the same.

- The in-service stress level has a statistically significant influence on the corrosion and C_T of PT strands. The C_T of stressed strands can be 1.6 to 17 percent less than the C_T of unstressed strands, provided other exposure conditions remain same.
- Probabilistic models relating the C_T of unstressed and stressed strands can be developed.
- The type of void, especially the GAS interface, has a statistically significant influence on the corrosion rate and resulting reduction in the tension capacity of strands. Typically, more localized corrosion will occur at strands in PT columns and anchorage zones on PT girders than at strands near the midspan region of PT girders.
- In fully grouted tendons (i.e., NV conditions), moisture and chlorides are the most influential factors in accelerating corrosion and reducing the C_T of the exposed strands. Hence, as long as the fully grouted tendons are protected from chloride and moisture ingress, the corrosion rate and capacity loss should be negligible. However, this protection is very difficult to accomplish, as cracks were found in both Class A and Class C grouts prepared in the laboratory. This is likely to be the case in the field, also. In addition, cracks or openings were found on PT ducts and anchorages in the field.
- BV, IV, and OV conditions have statistically similar effects on the C_T of strands. Because of the larger cathode-to-anode ratio, PV conditions facilitate a more corrosive environment than the BIOV condition, provided other exposure conditions remain similar.
- In general, the corrosivity of NV conditions is less than that of PV conditions and the corrosivity of PV conditions is less than that of BIOV conditions.
- In voided tendons, the presence of high moisture content or standing water seems to be the most influential factor in accelerating corrosion and reducing the C_T of the exposed strands. Hence, moisture should be prevented from infiltrating the ducts.
- When continuous-atmospheric (CA) and BIOV conditions exist, combinations of high relative humidity and temperature or the combinations of high relative humidity, temperature, and chlorides can cause severe corrosion in relatively short periods of time. When relative humidity and temperature are both low, the corrosion rates are low.
- When exposed to similar conditions, a seven-wire strand can lose more C_T than the sum of C_T lost by seven individual wires. A power

relationship between the C_T of strands and wires exhibits better accuracy than a linear relationship.

11.3.5. Structural reliability of post-tensioned bridges

Section 10 developed the structural reliability models for PT bridges. The time-variant strength and service reliabilities of a typical PT bridge were then assessed using the developed reliability models. The WD condition that is assumed for the analysis consisted of 2 months of wet time and 10 months of dry time in every year.

The conclusions derived from the study on strength reliability are:

- Strength reliability models can be developed and can be used to predict the flexural strength reliability index, $\beta_{strength}$, of PT bridges at future times.
- For the typical PT bridge (defined in Subsection 10.6.1) the $\beta_{strength}$ based on the parameters defined in Subsection 10.6.2 is as follows:
 - When all the strands are in “as-received” condition, $\beta_{strength}$ is above the β_{target} used for calibrating the AASHTO (2007) design code (i.e., 3.5) and recommended by ISO 13822 (2001) for the cases with low consequences of failure (i.e., 3.1).
 - If one tendon is exposed to WD cycles and completely corrodes or fails in tension due to high stress levels, the strength reliability model shows that the value of $\beta_{strength}$ stays above 3.5.
 - If two tendons are exposed to WD cycles and completely corrode or fail in tension due to high stress levels, the value of $\beta_{strength}$ drops below 3.5 but stays above 3.1.
 - When the bridge is subjected to HS20 loading and three or more external tendons are exposed to WD cycles with 0.006 percent chloride solution, the value of $\beta_{strength}$ drops below 3.5 within 25 years and below 3.1 within 35 years. These time estimates reduce to about 10 and 13 years when exposed to WD cycles with 1.8 percent chloride solution.

The conclusions derived from the study on service reliability are:

- Service reliability models can be developed and can be used to estimate the reliability index for maintaining the flexural stress limits for service load conditions, $\beta_{service}$, of PT bridges at future times.

- For the typical PT bridge (defined in Subsection 10.6.1), the $\beta_{strength}$ based on the parameters defined in Subsection 10.6.2 is as follows:
 - When the defined PT bridge is subjected to HS20 loading and all the strands are in “as-received” condition, the reliability model shows that value of $\beta_{service}$ is above 1.5 (i.e., the β_{target} value recommended by ISO 13822 [2001] for the cases with irreversible consequence of failure).
 - When the defined PT bridge is subjected to HL93 loading and all the strands are in “as-received” condition, the reliability model shows $\beta_{service}$ drops below 1.5. However, it stays above 0 (i.e., the β_{target} value recommended by ISO 13822 [2001] for the cases with reversible consequence of failure).
 - When the bridge is subjected to HS20 loading and only one tendon is exposed to WD cycles with 0.006 percent chloride solution, the reliability models show that $\beta_{service}$ can drop to a value below 1.5 within about 22 years. This time estimate reduces to about 10 years when exposed to 1.8 percent chloride solution.
 - When the bridge is subjected to HL93 loading and only one tendon is exposed to WD cycles with 0.006 percent chloride solution, $\beta_{service}$ can drop to a value below 1.5 within about 53 years. This time estimate reduces to about 10 years when exposed to 1.8 percent chloride solution.
 - Serviceability reduces significantly if more than one tendon is exposed to WD cycles.

11.4. RECOMMENDATIONS FOR FUTURE RESEARCH

Recommendations for future research include the following:

- Additional information on chloride exposure and chloride usage factors (such as the application rate of de-icing or anti-icing salts on bridges) in Texas should be collected and incorporated into the developed maps.
- A larger number of specimens should be tested to increase the statistical significance of the test results obtained. Also, experiments should be performed using more combinations of test variables.
- Additional modified ASTM G109 and bearing plate tests should be performed with more parameter combinations to better evaluate this potential issue.

- Information should be gathered from the field to better understand the exposure conditions and corrosion levels on strands in PT systems.
- Additional experiments should be conducted on unstressed strands with more combinations of test variables. The models for stressed strands developed in this research may then be re-calibrated using this new information on the C_T behavior of unstressed strands.
- The effect of the interface between the existing and repair grouts on corrosion and resulting loss in the C_T of strands should be investigated. The test specimens used in this research program could be modified to perform these tests.
- Strand surfaces inside the voided anchorages are inaccessible. This makes it difficult to estimate the level of corrosion at these void locations, and sufficient data on corrosion of strands in the field are not available. Special remote-controlled tools that can clean and collect the corrosion products at these strand surfaces need to be developed. Such tools can then be used to collect data on corrosion of strands in the field, especially under CA exposure conditions. These data could then be used in estimating the long-term performance of PT bridges.
- The following modifications should be incorporated into the structural reliability models developed in this research:
 - Develop probabilistic models to evaluate the moment demand due to continuous support conditions;
 - Develop probabilistic models to predict C_T of corroding strands in internal tendons;
 - Develop a model to more accurately assess the time-dependent prestress losses;
 - Develop models to assess reliability of girders with both bonded and unbonded tendons.
 - Develop a model to accommodate the effect of construction practices on structural behavior. These practices include sequential pre-stressing, type of construction technique (such as span-by-span, cantilever construction etc.), sequence in which the steel jackets supporting the segments are removed, and other influential factors.
- Collect field information on actual loading conditions and develop corresponding structural demand models.

- Extend the service reliability analysis performed with other service limit state criteria (tensile stress limit equal to 0, deflection limits, etc.).
- Develop structural reliability models based on shear, torsion, and other structural limit states.
- Develop a system reliability model for PT bridges based on the above recommended models for additional limit states.
- Assess the time-variant structural reliability of specific PT bridges in Texas.
- Assess the time-variant structural reliability of different standard AASHTO pre-tensioned bridges.

11.5. RECOMMENDATIONS FOR FIELD IMPLEMENTATIONS

Recommendations for implementations in the field include the following:

- For new bridges, consider coating the inside and outside surfaces of the bearing plates with epoxy or other dielectric material such that the galvanic corrosion between the bearing plate and other metallic materials in the PT systems can be minimized.
- Inspect the PT strands and girders at regular intervals, especially when the importance measures of the bridge and the likelihood of the occurrence of WD conditions inside the tendons are high.
- Prevent the infiltration of water and chlorides into PT systems and the formation of voids in PT systems during construction. Maintain dry conditions in the voids in tendons until the effects of grout repairs on strand corrosion can be assessed.

REFERENCES

- AASHTO, (1982), “AASHTO Bridge Design Specifications,” Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, (1992), “Standard specifications for highway bridges,” Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, (1994), “AASHTO LRFD Bridge Design Specifications,” Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, (1996), “AASHTO LRFD Bridge Design Specifications,” Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, (1998), “AASHTO LRFD Bridge Design Specifications,” Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, (1999), *Guide Specifications for design and construction of segmental concrete bridges*. (ISBN: 1-56051-103-6) American Association of State Highway and Transportation Officials, Washington D.C., USA.
- AASHTO, (2002), “Standard specifications for highway bridges,” 17th Edition, Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, (2004), “Standard specifications for highway bridges,” 19th Edition, Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, (2006), “Standard specifications for highway bridges,” 21th Edition, Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, (2007), “AASHTO LRFD Bridge Design Specifications,” Association of State Highway and Transportation Officials, Washington, D.C.
- ACI, (2003), “ACI 222R-03 – Protection of metals in concrete against corrosion,” ACI Committee 222, Document No. 22203, American Concrete Institute, Farmington Hills: MI.
- Akgul, F. and Frangopol, D.M., (2004), Lifetime Performance Analysis of Existing Prestressed Concrete Bridge Superstructures. *ASCE Journal of Structural Engineering*, 130(12):1889-1903.
- ASBI, (2000), “American segmental bridge institute grouting committee: Interim statement on grouting practices,” American Segmental Bridge Institute (ASBI), Phoenix, AZ, USA.
- Barton, K., (1980)., “Schutz gegen atmosphärische Korrosion - Theorie und Technik, Verlag Chemie.” *Protection of Metals, Z. Metallov*. Translator, 16, 387.

- Box, G.E.P. and Tiao, G.C., (1992), Bayesian Inference in Statistical Analysis. 113-122. Addison-Wesley: Reading: MA.
- Cavell, D.G. and Waldron, P., (2001), "*Parametric study of the residual strength of deteriorating simply-supported post-tensioned concrete bridges,*" Proceedings of the Institution of Civil Engineers, Vol. 11, pp. 341-352.
- CDCHB, (1991), Chinese Design Code for Highway Bridges, People's Communication Press, Beijing, China.
- Corvo, F., Minotas, J., Delgado, J., and Arroyave, C., (2005), "*Changes in atmospheric corrosion rate caused by chloride ions depending on rain regime,*" Corrosion Science, Vol. 47, pp. 883-892.
- Corvo, F., Perez, T., Martin, Y. Reyes, J., Dzib, L.R., Gonzalez-Sanchez, J., and Castaneda, A., (2008), "*Time of wetness in tropical climate: Considerations on the estimation of TOW according to ISO 9223 standard,*" Corrosion Science, Vol. 50, pp. 206-219.
- Cui, F., and Sagues, A., (2003), "*Corrosion performance of stainless steel clad rebar in simulated pore water and concrete,*" Corrosion 2003, NACE International, Houston, Paper No 03310.
- Dean S.W. and Reiser D.B., (2002). "Analysis of long-term atmospheric corrosion results from ISO CORRAG program," *Outdoor Atmospheric Corrosion*, ASTM STP 1421, Townsend, H.E., (Ed.). American Society of Testing and Materials (ASTM), West Conshohoken, Pennsylvania, 3-18.
- Decker, J.B., Rollins, K.M., and Ellsworth, J.C., (2008), "*Corrosion rate evaluation and prediction for piles based on long-term field performance,*" Journal of Geotechnical and Geoenvironmental Engineering, Vol. 134, No. 3, pp. 341-351.
- Der Kiureghian, A., Haukaas, T., and Fujimura, A., (2006). Structural reliability software at the University of California, Berkeley. *Structural Safety*, 28(1-2), 44-67.
- Diamond, S., (1981). "Effects of two Danish flyshes on alkali contents of pore solutions of cement-lyash pastes." *Cement and Concrete Research*, 11, 383-394.
- Ditlevsen, O., and Madsen, O.H., (1996), "Structural Reliability Methods. Monograph," (First edition published by John Wiley & Sons Ltd, Chichester.

- Drazic, D.M. and Vascic, V., (1989), “*The correlation between accelerated laboratory corrosion tests and atmospheric corrosion station tests on steels,*” *Corrosion Science*, Vol. 29, No. 10, pp. 1197-1204.
- Du, J.S., and Au, F.T.K., (2005), “*Deterministic and reliability analysis of prestressed concrete bridge girders: comparison of the Chinese, Hong Kong, and AASHTO LRFD Codes,*” *Structural Safety*, Vol. 27, pp. 230-245.
- Emmanuel, M., (1980), *Contemporary Architects*, St. Martin’s Press, New York.
- FDOT, (1999), “Corrosion Evaluation of Post-Tensioned Tendons on the Niles Channel Bridge,” FDOT Report, Florida Department of Transportation, Tallahassee, FL, USA.
- FDOT, (2001a), “Mid-Bay bridge post-tensioning evaluation – Final Report,” Corven Engineering, Inc., Florida Department of Transportation, Tallahassee, FL, USA.
- FDOT, (2001b), “Sunshine skyway bridge post-tensioned tendons investigation,” Parsons Brinckerhoff Quade and Douglas, Inc., Florida Department of Transportation, Tallahassee, FL, USA.
- FDOT, (2002), “Galvanic testing of stressed strands inside carbon steel pipe – Final report,” State Materials Office, Corrosion Research Laboratory, Florida Department of Transportation, Tallahassee, FL, USA.
- Feliu, S., Morcillo, M., and Feliu, S. Jr., (1993). “The prediction of atmospheric corrosion from meteorological and pollution parameters – 1. Annual corrosion.” *Corrosion Science*, 34(3), 403-414.
- FIB, (2001), “Durability of Post-tensioning tendons,” Technical Report, ISBN 978-2-88394-055-0, pp. 284, November 2001, Ghent, Belgium.
- Gardoni, P.G., Der Kiureghian, A., and Mosalam, K.M., (2002a), “Probabilistic Capacity Models and Fragility Estimates for Bridge Components and Systems,” PEER Report No. 2002/13, Pacific Earthquake Engineering Research Center, Department of Civil Engineering, University of California, Berkeley, CA, USA.
- Gardoni, P.G., Der Kiureghian, A., and Mosalam, K.M., (2002b), “Probabilistic Capacity Models and Fragility Estimates for Reinforced Concrete Columns based on Experimental Observations,” *ASCE Journal of Engineering Mechanics*, Vol. 128, No.10, pp. 1024-1038.
- Guttman, H. and Serada, P.J., (1968), *Metal Corrosion in the Atmosphere*, STP 435, American Society for Testing and Materials, PA.

- Hakkarainen, T., and Ylasaari, S., (1982). "Atmospheric corrosion testing in Finland." *Atmospheric Corrosion*, Ailor, W.H., (Ed.), John Wiley and Sons, New York, NY.
- Haldar, A. and Mahadevan, S., (2000), *Probability, Reliability and Statistical Methods in Engineering Design*.
- Hansen, B., (2007), "Forensic engineering: Tendon failure raises questions about grout in posttensioned bridges," *Civil Engineering News*, November, pp. 17-18.
- Haynie, F.H., and Upham, J.B., (1974), Correlation between corrosion behavior of steel and atmospheric pollution data," ASTM STP 558, American Society of Testing and Materials, pp. 33-51.
- Hognestad, E. A., (1951), "A study of combined bending and axial load in reinforced concrete members." Bulletin Series No. 399, University of Illinois, Engineering Experiment Station, Urbana, Ill.
- ISO 13822, (2001), Bases for Bases for design of structures -- Assessment of existing structures, International Standards, Organization, Switzerland.
- Jones, D. A., (1996), *Principles and Prevention of Corrosion*, 2nd Ed., Prentice Hall, Inc., NJ.
- Klinesmith, D.E., McCuen, R.H., and Albrecht, P., (2007), "*Effect of Environmental Conditions on Corrosion Rates*," *Journal of Materials in Civil Engineering*, Vol. 19, No. 2, pp. 121-129.
- Kobus, J., (2000), "*Long-term atmospheric corrosion monitoring*," *Materials and Corrosion*, Vol. 51, pp. 104-108.
- Knotkova, D., Gullman, J., Holler, P., Kucera, V., (1984). "Assessment of corrosivity by short-term atmospheric field tests of technically important materials." *Proceedings of the 9th International Congress on Metallic Corrosion*", National Research Council of Canada, Toronto, 198-205.
- Kovač J., Leban, M., and Legat, A., (2007), "Detection of SCC on prestressing steel wire by the simultaneous use of electrochemical noise and acoustic emission measurements," *Electrochimica Acta*, Vol. 52, pp. 7607-7616.
- Laco, J.I.I, Leisa Mestres, F., Cadena Villota, F., and Bilurnina Alter, L., (2004), "*Urban and marine corrosion: Comparative behavior between field and laboratory conditions*," *Materials and Corrosion*, Vol. 55, No. 9, pp. 689-694.

- Liu P-L, Lin H-Z, and Der Kiureghian, A., (1989), CalREL user manual. Report No. UCB/SEMM-89/18, Structural Engineering, Mechanics and Materials, Department of Civil Engineering, University of California, Berkeley, CA, April.
- Mammoliti, L.T., Brown, L.C., Hasson, C.M., and Hope, B.B., (1996), “*The influence of surface finish of reinforcing steel and pH of the test solution on the chloride threshold concentration for corrosion initiation in synthetic pore solutions,*” *Cement and Concrete Research*, Vol. 26, No. 4, pp. 545-550.
- Mander, J. B., Priestley, M. J. N. and Park, R., (1988a). “Theoretical stress-strain model for confined concrete.” *ASCE Journal of Structural Engineering*, 114(8), 1804-1826.
- Mander, J. B., Priestley, M. J. N. and Park, R., (1988b). “Observed stress-strain behavior of confined concrete.” *ASCE Journal of Structural Engineering*, 114(8), 1827-1849.
- Mansfield, F., (1979), “*Atmospheric corrosion rates, time-of-wetness, and relative humidity,*” *Werkstoffe und Korrosion*, Vol. 30, pp. 38-42.
- McCuen, R.H., Albrecht, P, and Cheng, J., (1992). “A new approach to power-model regression of corrosion penetration data.” *Corrosion and Control for Infrastructure*, ASTM STP 1137, American Society of Testing and Materials, West Conshohoken, PA.
- McGray, J.P., (1998). “North American free trade agreement truck highway corridors: US-Mexico truck rivers of trade.” *Transportation Research Record-1613*, Paper No. 98-0757, 71-78.
- Melchers, R.E., (1998), “*Probabilistic modeling of immersion marine corrosion,*” *Structural Safety and Reliability*, Vol. 3, pp. 1143-1149.
- Melchers, R.E., (1999), “*Corrosion uncertainty modeling for steel structures,*” *J. Constructional Steel Res.*, Vol. 52, pp. 3-19.
- Naaman, A.E., (2004), “*Prestressed concrete analysis and design,*” 2nd Edition, Techno Press, Ann Arbor, MI.
- Naaman, A.E., and Alkhalil, F.M., (1992), “Stress at Ultimate in Unbonded Post-Tensioning Tendons: Part 2--Proposed Methodology, *ACI Structural Journal*, Vol. 88, No. 6, pp. 683-692.
- Natesan, M., Venkatachari, G., and Palaniswamy, N., (2006), “*Kinetics of atmospheric corrosion of mild steel, zinc, galvanized iron and aluminium at 10 exposure stations in India,*” *Corrosion Science*, Vol. 48, pp. 3584-3608.

- Nawy, E.G., (2003), "Prestressed concrete – A fundamental approach," 4th Edition, Prentice Hall, Saddle River, NJ.
- NCDC, (2009), National Climatic Data Center, www.ncdc.noaa.gov, Last accessed on November 1, 2009,
- NCHRP, (1998), "Durability of precast segmental bridges," NCHRP Web Document No. 15, Project 20-7/Task 92 (Editors: Poston, R.W. & Wouters, J.P.), National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, D.C., USA.
- NCHRP, (2003), "Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders," National Cooperative Highway Research Program (NCHRP): Transportation Research Board: National Research Council: Washington: D.C.
- Neville, A. M., (1998), *Properties of Concrete*, 4th Ed., John Wiley & Sons, Inc., New York.
- Nowak, A. S, and Collins, K. R., (2000), *Reliability of Structures*, McGraw-Hill Companies, Inc.
- PCI, (1999), *PCI design handbook*, Precast/Prestressed Concrete Institute, Chicago, Illinois.
- Perret, S., Khayat, K.H., Gagnon, E., and Rhazi, J., (2002), "Repair of 130-year old masonry bridge using high-performance cement grout," *ASCE Journal of Bridge Engineering*, Vol. 7, No. 1, pp. 31-38.
- Pillai, R.G. and Trejo, D., (2005), "*Surface reinforcement effects on critical chloride threshold of steel reinforcement*," *ACI Materials Journal*, Vol. 102, No. 2, pp. 103-109.
- Poston, W.R., Frank, K.H., and West, J.S., (2003), "Enduring strength," *Civil Engineering*, American Society of Civil Engineering, 73(9), pp.58-63.
- Proverbio, E., and Longo, P., (2003), "Failure mechanisms of high strength steels in bicarbonate solutions under anodic polarization," *Corrosion Science*, Vol. 45, pp. 2017-2030.
- PTI, (1998), "Acceptance Standards for Post-tensioning Applications, Post-tensioning Institute, Pheonix, AR.
- PTI, (2003), "Specification for grouting of post-tensioned structures," Post-Tensioning Institute (PTI), Phoenix, AZ, USA.
- Roadstothe future, (2009), http://www.roadstothe future.com/I295_VA_JRB.jpg, Last accessed on January 28, 2009.
- Roberge, P.R., (1999), "Handbook of Corrosion Engineering," McGraw-Hill Book Company, New York, NY.

- Romanoff, M., (1962), “*Corrosion of steel pilings in soils*,” NBS Monograph No. 58, National Bureau of Standards, U.S. Dept. of Commerce, Washington, D.C.
- Ryall, K., Wollmann, G.P., Breen, J.E., and Kreger, M.E., (1993). “Fretting fatigue of external tendons in segmental bridge deviators.” *Proceedings of the Workshop on Behavior of External Prestressing in Structures.*, Association Francaise Pour la Construction, Saint-Remy-les-Chevreuse, France, 131-140.
- Ryan, T.P., (2007), “*Modern Engineering Statistics*, Wiley Interscience, Hoboken, New Jersey.
- Sagues, A., (2003), “Durability Research,” <http://www.dot.state.fl.us/statematerialsoffice/administration/meetings/dme/presentation/corrosiondurability.pdf>, Last accessed on February 4, 2008.
- Sanchez, J., Fulla, J., Andrade, C., and Alonso, C., (2007), “Stress corrosion cracking mechanism of prestressing steels in bicarbonate solutions,” *Corrosion Science*, Vol. 49, July, pp. 4069-4080.
- Sanchez, J., Trejo, D., and Hueste, M.D., (2003), “Personal communication on possible corrosion mechanisms in PT bridges in Texas,”
- Sason, A. S., (1992), “Evaluation of degree of rusting on prestressed concrete strand,” *PCI Journal*, May–June, pp. 25-30.
- Schokker, A. J., Koester, B.D., Breen, J. E., and Kreger, M. E., (1999), “Development of high-performance grouts for bonded post-tensioned structures,” TxDOT Report 0-1405-2, Center for Transportation Research, The University of Texas, Austin, TX.
- Schokker, A. J., Breen, J. E., and Kreger, M. E. (2002), “Simulated field testing of high-performance grouts for post-tensioning.” *Journal of Bridge Engineering*, 7(2), 127-133.
- Schupack, M., (2004), “PT grout: Bleedwater voids,” *Concrete International*, 26(8), pp. 69-77.
- SDMHR, (1997), *Structures Design Manual for Highways and Railways*, Highways Department, Government of the Hong Kong Special Administrative Region, 2nd ed., with Amendment No. 1/2002, Hong Kong.
- SHRP-S/FR-92-110, (1992), *Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion-Volume 8: Procedure Manual*, “Standard Test Method for Chloride Content in Concrete Using the Specific Ion Probe”, Strategic Highway Research Program, Washington, DC, pp. 85-105.

- Southwell, C.R., Bultman, J.D., and Hummer, C.W. Jr., (1979), “*Estimation of service life of steel in seawater*,” Seawater corrosion handbook, New Jersey, Noyes Data Corporation, pp. 374-387.
- Structurae, (2009), <http://en.structurae.de/photos/index.cfm?JS=15986>, Last accessed on January 28, 2009.
- Tabsh, S.W., (1995), Structural safety of post-tensioned concrete slab bridges with unbonded tendons. *ACI Structural Journal*, 92(4):1-7.
- Ting, S-C. and Nowak, A.S., (1991), Effect of tendon-area loss on flexural behavior of P/C beams. *ASCE Journal of Structural Engineering*, 117(4):1127-1143.
- Todeschini, C. E., Bianchini, A. C., and Kesler, C. E., (1964), “Behaviour of concrete columns reinforced with high strength steels.” *ACI J.*, 61~67, 701-716.
- Trejo, D. and Pillai, R.G., (2003), “*Accelerated chloride threshold testing: Part 1 – ASTM A 615 and A 706 Reinforcement*,” *ACI Materials Journal*, Vol. 100, No. 6, pp. 519-527.
- TRRL, (1987), “Seventh Report of the Committee for the two years ending July 1987.” EA/88/4, Standing Committee on Structural Safety, Transport and Road Research Laboratory, UK.
- TxDOT, (2004), “Critical evaluation and condition assessment of post-tensioned bridges in Texas – Final Report.” Texas Department of Transportation, Austin, TX.
- West, J.S, Larosche, C.J., Koester, B.D., Breen, J.E., and Kreger, M.E., (1999), “State-of-the-Art Report about durability of post-tensioned bridge substructures,” TxDOT Research Report 0-1405-1, Center for Transportation Research, The University of Texas, Austin, TX.
- Wikipedia, (2009), <http://en.wikipedia.org/wiki/Image:Sunsky.jpg>, Last accessed on January 28, 2009.
- Wollmann, R. C.L., Breen, J.E., and Kreger, M.E., (2001), “*Live load tests on the San Antonio “Y”*,” *Journal of Bridge Engineering*, Vol. 6, No. 6, pp. 556-563.
- Woodward, R. J., (1981), “Conditions within ducts in post-tensioned prestressed concrete bridges.” LR 980, Transport and Road Research Laboratory, Department of Transport, Crowthorne, Berkshire, UK.
- Woodward, R., Cullington, D., and Lane, J., (2001), “Strategies for the management of post-tensioned concrete bridges.” *Current and Future Trends in Bridge Design, Construction, and Maintenance 2: Safety, Economy, Sustainability, and Aesthetics*, Hong Kong, 23-32.

Worldbreak, (2009), <http://watertowers.worldbreak.com/ChoctawhatcheeMidBay.jpg>, Last accessed on January 28, 2009.

YouTube, (2009), http://www.youtube.com/watch?v=PUg_EqQN6Gg&feature=related, Last accessed on January 28, 2009.

APPENDIX A. ALL TENSION CAPACITY DATA

Table A.1. Tension Capacity of Unstressed Strands Exposed to WD Conditions.

Sample ID	Tensile strength (kips)	Chloride (%Cl-)	Void type	Exposure time (months)
531	59.62	0.0001	No corrosion	0.03
532	59.36	0.0001	No corrosion	0.03
533	59.01	0.0001	No corrosion	0.03
601	59.29	0.0001	No corrosion	0.03
602	59.16	0.0001	No corrosion	0.03
603	58.83	0.0001	No corrosion	0.03
521	59.16	0.0001	No corrosion	0.03
522	59.54	0.0001	No corrosion	0.03
523	59.41	0.0001	No corrosion	0.03
591	59.33	0.0001	No corrosion	0.03
592	59.32	0.0001	No corrosion	0.03
593	59.58	0.0001	No corrosion	0.03
524	58.49	0.0001	No corrosion	0.03
525	58.99	0.0001	No corrosion	0.03
526	59.09	0.0001	No corrosion	0.03
594	59.03	0.0001	No corrosion	0.03
595	58.93	0.0001	No corrosion	0.03
596	59.12	0.0001	No corrosion	0.03
534	59.64	0.0001	No corrosion	0.03
535	59.59	0.0001	No corrosion	0.03
536	59.45	0.0001	No corrosion	0.03
604	59.54	0.0001	No corrosion	0.03
605	59.40	0.0001	No corrosion	0.03
606	59.56	0.0001	No corrosion	0.03
541	58.59	0.0060	No Void	12
542	58.93	0.0060	No Void	12
543	58.92	0.0060	No Void	12
544	62.11	0.0060	No Void	12
545	59.17	0.0060	No Void	12
611	58.79	0.0060	No Void	12
612	59.34	0.0060	No Void	12
613	58.98	0.0060	No Void	12
614	58.41	0.0060	No Void	12
615	59.20	0.0060	No Void	12
561	59.39	0.0180	No Void	12
562	58.76	0.0180	No Void	12
563	58.43	0.0180	No Void	12
564	58.89	0.0180	No Void	12
565	59.07	0.0180	No Void	12
631	59.29	0.0180	No Void	12

632	58.71	0.0180	No Void	12
633	59.29	0.0180	No Void	12
634	58.94	0.0180	No Void	12
635	59.05	0.0180	No Void	12
571	59.55	0.1800	No Void	12
572	59.64	0.1800	No Void	12
573	62.39	0.1800	No Void	12
574	58.78	0.1800	No Void	12
575	59.26	0.1800	No Void	12
641	58.59	0.1800	No Void	12
642	58.93	0.1800	No Void	12
643	58.92	0.1800	No Void	12
644	62.11	0.1800	No Void	12
645	59.17	0.1800	No Void	12
581	53.99	1.8000	No Void	12
582	57.86	1.8000	No Void	12
583	58.20	1.8000	No Void	12
584	59.08	1.8000	No Void	12
585	54.49	1.8000	No Void	12
651	57.39	1.8000	No Void	12
652	55.56	1.8000	No Void	12
653	56.48	1.8000	No Void	12
654	56.77	1.8000	No Void	12
655	54.71	1.8000	No Void	12
546	59.55	0.0060	No Void	21
547	59.61	0.0060	No Void	21
548	58.78	0.0060	No Void	21
549	59.67	0.0060	No Void	21
550	59.31	0.0060	No Void	21
616	59.27	0.0060	No Void	21
617	58.19	0.0060	No Void	21
618	57.95	0.0060	No Void	21
619	59.03	0.0060	No Void	21
620	58.85	0.0060	No Void	21
566	58.90	0.0180	No Void	21
567	59.33	0.0180	No Void	21
568	59.33	0.0180	No Void	21
569	58.84	0.0180	No Void	21
570	59.18	0.0180	No Void	21
636	58.80	0.0180	No Void	21
637	59.13	0.0180	No Void	21
638	59.33	0.0180	No Void	21
639	58.83	0.0180	No Void	21
640	59.18	0.0180	No Void	21
576	58.21	0.1800	No Void	21

577	57.14	0.1800	No Void	21
578	57.58	0.1800	No Void	21
579	58.48	0.1800	No Void	21
580	59.25	0.1800	No Void	21
646	59.47	0.1800	No Void	21
647	58.56	0.1800	No Void	21
648	59.12	0.1800	No Void	21
649	58.73	0.1800	No Void	21
650	58.27	0.1800	No Void	21
586	52.58	1.8000	No Void	21
587	57.28	1.8000	No Void	21
588	51.01	1.8000	No Void	21
589	56.81	1.8000	No Void	21
590	53.59	1.8000	No Void	21
656	56.92	1.8000	No Void	21
657	55.92	1.8000	No Void	21
658	51.82	1.8000	No Void	21
659	57.08	1.8000	No Void	21
660	57.30	1.8000	No Void	21
431	57.91	0.0060	Bleedwater	12
432	58.13	0.0060	Bleedwater	12
433	58.74	0.0060	Bleedwater	12
434	58.47	0.0060	Bleedwater	12
435	57.62	0.0060	Bleedwater	12
481	58.90	0.0060	Bleedwater	12
482	57.56	0.0060	Bleedwater	12
483	55.20	0.0060	Bleedwater	12
484	58.53	0.0060	Bleedwater	12
485	58.92	0.0060	Bleedwater	12
451	56.40	0.0180	Bleedwater	12
452	56.55	0.0180	Bleedwater	12
453	57.27	0.0180	Bleedwater	12
454	56.90	0.0180	Bleedwater	12
455	58.12	0.0180	Bleedwater	12
501	57.45	0.0180	Bleedwater	12
502	57.91	0.0180	Bleedwater	12
503	58.03	0.0180	Bleedwater	12
504	58.63	0.0180	Bleedwater	12
505	57.25	0.0180	Bleedwater	12
461	54.67	1.8000	Bleedwater	12
462	54.42	1.8000	Bleedwater	12
463	55.48	1.8000	Bleedwater	12
464	54.57	1.8000	Bleedwater	12
465	54.27	1.8000	Bleedwater	12
511	56.04	1.8000	Bleedwater	12

512	54.14	1.8000	Bleedwater	12
513	55.75	1.8000	Bleedwater	12
514	54.28	1.8000	Bleedwater	12
515	50.21	1.8000	Bleedwater	12
436	54.83	0.0060	Bleedwater	21
437	55.43	0.0060	Bleedwater	21
438	54.24	0.0060	Bleedwater	21
439	57.69	0.0060	Bleedwater	21
440	56.61	0.0060	Bleedwater	21
486	55.34	0.0060	Bleedwater	21
487	57.23	0.0060	Bleedwater	21
488	56.16	0.0060	Bleedwater	21
489	55.84	0.0060	Bleedwater	21
490	58.02	0.0060	Bleedwater	21
456	55.49	0.0180	Bleedwater	21
457	51.30	0.0180	Bleedwater	21
458	53.89	0.0180	Bleedwater	21
459	53.75	0.0180	Bleedwater	21
460	52.83	0.0180	Bleedwater	21
506	53.92	0.0180	Bleedwater	21
507	55.17	0.0180	Bleedwater	21
508	54.99	0.0180	Bleedwater	21
509	53.68	0.0180	Bleedwater	21
510	54.38	0.0180	Bleedwater	21
466	48.35	1.8000	Bleedwater	21
467	49.79	1.8000	Bleedwater	21
468	45.74	1.8000	Bleedwater	21
469	49.46	1.8000	Bleedwater	21
470	43.79	1.8000	Bleedwater	21
516	42.78	1.8000	Bleedwater	21
517	52.25	1.8000	Bleedwater	21
518	41.20	1.8000	Bleedwater	21
519	46.18	1.8000	Bleedwater	21
520	42.91	1.8000	Bleedwater	21
301	59.92	0.0060	Inclined	12
302	58.59	0.0060	Inclined	12
303	58.34	0.0060	Inclined	12
304	56.01	0.0060	Inclined	12
305	56.53	0.0060	Inclined	12
371	56.34	0.0060	Inclined	12
373	57.43	0.0060	Inclined	12
374	57.67	0.0060	Inclined	12
375	56.47	0.0060	Inclined	12
321	54.09	0.0180	Inclined	12
322	57.77	0.0180	Inclined	12

323	58.82	0.0180	Inclined	12
324	55.80	0.0180	Inclined	12
325	57.11	0.0180	Inclined	12
391	53.80	0.0180	Inclined	12
392	54.99	0.0180	Inclined	12
393	57.61	0.0180	Inclined	12
394	56.11	0.0180	Inclined	12
395	55.61	0.0180	Inclined	12
331	54.22	0.1800	Inclined	12
332	53.50	0.1800	Inclined	12
333	53.92	0.1800	Inclined	12
334	50.81	0.1800	Inclined	12
335	56.14	0.1800	Inclined	12
401	54.00	0.1800	Inclined	12
402	54.09	0.1800	Inclined	12
403	53.69	0.1800	Inclined	12
404	55.76	0.1800	Inclined	12
405	52.56	0.1800	Inclined	12
341	55.80	1.8000	Inclined	12
342	55.96	1.8000	Inclined	12
343	57.95	1.8000	Inclined	12
344	54.91	1.8000	Inclined	12
345	54.15	1.8000	Inclined	12
411	57.37	1.8000	Inclined	12
412	58.45	1.8000	Inclined	12
413	53.78	1.8000	Inclined	12
414	53.38	1.8000	Inclined	12
415	55.03	1.8000	Inclined	12
306	57.72	0.0060	Inclined	21
307	56.78	0.0060	Inclined	21
308	55.11	0.0060	Inclined	21
309	56.48	0.0060	Inclined	21
310	55.63	0.0060	Inclined	21
376	54.88	0.0060	Inclined	21
377	54.89	0.0060	Inclined	21
378	54.75	0.0060	Inclined	21
379	52.47	0.0060	Inclined	21
380	53.54	0.0060	Inclined	21
326	55.32	0.0180	Inclined	21
327	52.27	0.0180	Inclined	21
328	47.21	0.0180	Inclined	21
329	53.71	0.0180	Inclined	21
330	51.75	0.0180	Inclined	21
396	51.85	0.0180	Inclined	21
397	50.45	0.0180	Inclined	21

398	52.92	0.0180	Inclined	21
399	55.86	0.0180	Inclined	21
400	55.09	0.0180	Inclined	21
336	50.81	0.1800	Inclined	21
337	51.64	0.1800	Inclined	21
338	49.01	0.1800	Inclined	21
339	53.89	0.1800	Inclined	21
340	51.97	0.1800	Inclined	21
406	49.17	0.1800	Inclined	21
407	52.23	0.1800	Inclined	21
408	45.97	0.1800	Inclined	21
409	47.98	0.1800	Inclined	21
410	52.39	0.1800	Inclined	21
346	48.90	1.8000	Inclined	21
347	43.06	1.8000	Inclined	21
348	45.94	1.8000	Inclined	21
349	47.48	1.8000	Inclined	21
350	44.02	1.8000	Inclined	21
416	47.01	1.8000	Inclined	21
417	44.02	1.8000	Inclined	21
418	45.60	1.8000	Inclined	21
419	44.57	1.8000	Inclined	21
420	44.20	1.8000	Inclined	21
21	60.49	0.0060	Orthogonal	12
22	58.26	0.0060	Orthogonal	12
23	56.56	0.0060	Orthogonal	12
24	57.31	0.0060	Orthogonal	12
25	54.23	0.0060	Orthogonal	12
91	56.61	0.0060	Orthogonal	12
92	57.46	0.0060	Orthogonal	12
93	54.98	0.0060	Orthogonal	12
94	55.03	0.0060	Orthogonal	12
95	56.54	0.0060	Orthogonal	12
41	58.66	0.0180	Orthogonal	12
42	57.51	0.0180	Orthogonal	12
43	57.23	0.0180	Orthogonal	12
44	58.84	0.0180	Orthogonal	12
45	59.22	0.0180	Orthogonal	12
111	55.30	0.0180	Orthogonal	12
112	55.57	0.0180	Orthogonal	12
113	61.37	0.0180	Orthogonal	12
114	58.52	0.0180	Orthogonal	12
115	54.52	0.0180	Orthogonal	12
51	53.21	0.1800	Orthogonal	12
52	52.70	0.1800	Orthogonal	12

53	52.77	0.1800	Orthogonal	12
54	55.44	0.1800	Orthogonal	12
55	57.25	0.1800	Orthogonal	12
121	53.34	0.1800	Orthogonal	12
122	53.39	0.1800	Orthogonal	12
123	53.63	0.1800	Orthogonal	12
124	53.91	0.1800	Orthogonal	12
125	53.90	0.1800	Orthogonal	12
64	53.99	1.8000	Orthogonal	12
65	55.86	1.8000	Orthogonal	12
66	54.90	1.8000	Orthogonal	12
131	55.57	1.8000	Orthogonal	12
132	51.64	1.8000	Orthogonal	12
133	54.89	1.8000	Orthogonal	12
134	54.18	1.8000	Orthogonal	12
135	53.41	1.8000	Orthogonal	12
26	53.66	0.0060	Orthogonal	21
27	54.47	0.0060	Orthogonal	21
28	52.92	0.0060	Orthogonal	21
29	54.21	0.0060	Orthogonal	21
30	53.08	0.0060	Orthogonal	21
96	52.35	0.0060	Orthogonal	21
97	53.38	0.0060	Orthogonal	21
98	52.56	0.0060	Orthogonal	21
99	56.09	0.0060	Orthogonal	21
100	54.35	0.0060	Orthogonal	21
46	52.14	0.0180	Orthogonal	21
47	50.69	0.0180	Orthogonal	21
48	53.19	0.0180	Orthogonal	21
49	53.81	0.0180	Orthogonal	21
50	53.39	0.0180	Orthogonal	21
116	50.67	0.0180	Orthogonal	21
117	51.58	0.0180	Orthogonal	21
119	51.37	0.0180	Orthogonal	21
56	49.36	0.1800	Orthogonal	21
57	52.52	0.1800	Orthogonal	21
58	52.72	0.1800	Orthogonal	21
59	49.60	0.1800	Orthogonal	21
60	47.66	0.1800	Orthogonal	21
126	44.95	0.1800	Orthogonal	21
127	49.49	0.1800	Orthogonal	21
128	47.17	0.1800	Orthogonal	21
129	53.72	0.1800	Orthogonal	21
130	46.24	0.1800	Orthogonal	21
67	46.31	1.8000	Orthogonal	21

68	43.03	1.8000	Orthogonal	21
69	42.93	1.8000	Orthogonal	21
70	43.98	1.8000	Orthogonal	21
136	44.85	1.8000	Orthogonal	21
137	45.69	1.8000	Orthogonal	21
138	45.00	1.8000	Orthogonal	21
139	46.48	1.8000	Orthogonal	21
140	45.57	1.8000	Orthogonal	21
161	59.63	0.0060	Parallel	12
162	58.84	0.0060	Parallel	12
163	59.14	0.0060	Parallel	12
164	58.96	0.0060	Parallel	12
165	59.37	0.0060	Parallel	12
231	59.48	0.0060	Parallel	12
232	58.52	0.0060	Parallel	12
233	59.29	0.0060	Parallel	12
234	59.24	0.0060	Parallel	12
235	63.29	0.0060	Parallel	12
181	59.02	0.0180	Parallel	12
182	59.20	0.0180	Parallel	12
183	59.04	0.0180	Parallel	12
184	59.33	0.0180	Parallel	12
185	59.04	0.0180	Parallel	12
251	59.58	0.0180	Parallel	12
252	58.48	0.0180	Parallel	12
253	59.16	0.0180	Parallel	12
254	58.43	0.0180	Parallel	12
255	59.50	0.0180	Parallel	12
191	63.04	0.1800	Parallel	12
192	59.09	0.1800	Parallel	12
193	58.79	0.1800	Parallel	12
194	62.85	0.1800	Parallel	12
195	58.76	0.1800	Parallel	12
261	58.96	0.1800	Parallel	12
262	58.22	0.1800	Parallel	12
263	59.12	0.1800	Parallel	12
264	58.11	0.1800	Parallel	12
265	58.63	0.1800	Parallel	12
204	61.95	1.8000	Parallel	12
205	58.48	1.8000	Parallel	12
210	58.34	1.8000	Parallel	12
271	57.98	1.8000	Parallel	12
272	58.67	1.8000	Parallel	12
273	57.79	1.8000	Parallel	12
274	58.03	1.8000	Parallel	12

275	58.51	1.8000	Parallel	12
166	58.27	0.0060	Parallel	21
167	58.95	0.0060	Parallel	21
168	57.68	0.0060	Parallel	21
169	58.54	0.0060	Parallel	21
170	57.68	0.0060	Parallel	21
236	59.00	0.0060	Parallel	21
237	58.59	0.0060	Parallel	21
238	58.78	0.0060	Parallel	21
239	59.13	0.0060	Parallel	21
240	58.72	0.0060	Parallel	21
186	58.52	0.0180	Parallel	21
187	57.86	0.0180	Parallel	21
188	56.94	0.0180	Parallel	21
189	57.87	0.0180	Parallel	21
190	57.47	0.0180	Parallel	21
256	59.13	0.0180	Parallel	21
257	56.97	0.0180	Parallel	21
259	57.91	0.0180	Parallel	21
260	57.59	0.0180	Parallel	21
196	56.81	0.1800	Parallel	21
197	54.69	0.1800	Parallel	21
198	56.08	0.1800	Parallel	21
199	56.72	0.1800	Parallel	21
200	56.06	0.1800	Parallel	21
266	54.37	0.1800	Parallel	21
267	55.40	0.1800	Parallel	21
268	52.14	0.1800	Parallel	21
270	55.38	0.1800	Parallel	21
206	53.62	1.8000	Parallel	21
207	52.27	1.8000	Parallel	21
208	54.31	1.8000	Parallel	21
209	55.25	1.8000	Parallel	21
276	48.36	1.8000	Parallel	21
277	53.67	1.8000	Parallel	21
278	53.17	1.8000	Parallel	21
279	53.55	1.8000	Parallel	21
280	54.28	1.8000	Parallel	21

Table A.2. Tension Capacity Data from Stressed Strand Samples Exposed to WD Conditions.

Sample ID	Tensile strength (kips)	Chloride (%Cl-)	Void type	Exposure time (months)
531	59.62	0.0001	No corrosion	0.03
532	59.36	0.0001	No corrosion	0.03
533	59.01	0.0001	No corrosion	0.03
601	59.29	0.0001	No corrosion	0.03
602	59.16	0.0001	No corrosion	0.03
603	58.83	0.0001	No corrosion	0.03
521	59.16	0.0001	No corrosion	0.03
522	59.54	0.0001	No corrosion	0.03
523	59.41	0.0001	No corrosion	0.03
591	59.33	0.0001	No corrosion	0.03
592	59.32	0.0001	No corrosion	0.03
593	59.58	0.0001	No corrosion	0.03
524	58.49	0.0001	No corrosion	0.03
525	58.99	0.0001	No corrosion	0.03
526	59.09	0.0001	No corrosion	0.03
594	59.03	0.0001	No corrosion	0.03
595	58.93	0.0001	No corrosion	0.03
596	59.12	0.0001	No corrosion	0.03
534	59.64	0.0001	No corrosion	0.03
535	59.59	0.0001	No corrosion	0.03
536	59.45	0.0001	No corrosion	0.03
604	59.54	0.0001	No corrosion	0.03
605	59.40	0.0001	No corrosion	0.03
606	59.56	0.0001	No corrosion	0.03
S-541	59.40	0.0060	No Void	12
S-542	59.47	0.0060	No Void	12
S-543	59.22	0.0060	No Void	16
S-546	59.44	0.0060	No Void	16
S-547	59.47	0.0060	No Void	21
S-548	59.43	0.0060	No Void	21
S-611	59.36	0.0060	No Void	12
S-612	59.35	0.0060	No Void	12
S-613	59.17	0.0060	No Void	16
S-616	59.34	0.0060	No Void	16
S-617	59.70	0.0060	No Void	21
S-618	59.29	0.0060	No Void	21
S-561	59.53	0.0180	No Void	12

S-562	59.42	0.0180	No Void	12
S-563	59.49	0.0180	No Void	16
S-566	59.55	0.0180	No Void	16
S-567	59.63	0.0180	No Void	21
S-568	59.95	0.0180	No Void	21
S-631	59.26	0.0180	No Void	12
S-632	59.31	0.0180	No Void	12
S-633	59.67	0.0180	No Void	16
S-636	59.49	0.0180	No Void	16
S-637	58.72	0.0180	No Void	21
S-638	58.63	0.0180	No Void	21
S-581	55.22	1.8000	No Void	12
S-582	54.66	1.8000	No Void	12
S-583	52.42	1.8000	No Void	16
S-586	52.66	1.8000	No Void	16
S-587	54.73	1.8000	No Void	21
S-588	53.28	1.8000	No Void	21
S-651	57.69	1.8000	No Void	12
S-652	58.10	1.8000	No Void	12
S-653	55.83	1.8000	No Void	16
S-657	54.10	1.8000	No Void	21
S-658	53.86	1.8000	No Void	21
S-021	55.51	0.0060	Orthogonal	12
S-022	50.98	0.0060	Orthogonal	16
S-023	54.53	0.0060	Orthogonal	12
S-024	53.32	0.0060	Orthogonal	12
S-025	50.93	0.0060	Orthogonal	16
S-026	53.12	0.0060	Orthogonal	16
S-028	51.56	0.0060	Orthogonal	21
S-029	49.86	0.0060	Orthogonal	21
S-030	50.46	0.0060	Orthogonal	21
S-091	52.65	0.0060	Orthogonal	12
S-092	50.78	0.0060	Orthogonal	16
S-093	53.01	0.0060	Orthogonal	12
S-094	52.65	0.0060	Orthogonal	12
S-095	49.02	0.0060	Orthogonal	16
S-096	49.78	0.0060	Orthogonal	16
S-097	47.20	0.0060	Orthogonal	21
S-099	48.36	0.0060	Orthogonal	21
S-041	53.68	0.0180	Orthogonal	12
S-042	45.25	0.0180	Orthogonal	16

S-043	51.69	0.0180	Orthogonal	12
S-044	52.60	0.0180	Orthogonal	12
S-047	46.31	0.0180	Orthogonal	21
S-048	46.15	0.0180	Orthogonal	21
S-049	47.17	0.0180	Orthogonal	21
S-050	46.59	0.0180	Orthogonal	21
S-111	50.29	0.0180	Orthogonal	12
S-112	47.76	0.0180	Orthogonal	16
S-113	51.18	0.0180	Orthogonal	12
S-114	50.99	0.0180	Orthogonal	12
S-115	43.87	0.0180	Orthogonal	16
S-116	47.98	0.0180	Orthogonal	16
S-117	45.69	0.0180	Orthogonal	21
S-118	45.00	0.0180	Orthogonal	21
S-119	45.29	0.0180	Orthogonal	21
S-120	47.07	0.0180	Orthogonal	21
S-061	43.18	1.8000	Orthogonal	12
S-062	44.02	1.8000	Orthogonal	12
S-063	43.52	1.8000	Orthogonal	12
S-064	43.52	1.8000	Orthogonal	16
S-065	46.80	1.8000	Orthogonal	16
S-066	51.00	1.8000	Orthogonal	16
S-067	35.94	1.8000	Orthogonal	21
S-068	38.12	1.8000	Orthogonal	21
S-069	37.13	1.8000	Orthogonal	21
S-131	45.96	1.8000	Orthogonal	12
S-132	31.49	1.8000	Orthogonal	16
S-133	44.44	1.8000	Orthogonal	12
S-134	43.85	1.8000	Orthogonal	12
S-135	36.59	1.8000	Orthogonal	16
S-136	36.73	1.8000	Orthogonal	16
S-137	26.75	1.8000	Orthogonal	21
S-138	35.08	1.8000	Orthogonal	21
S-139	34.45	1.8000	Orthogonal	21
S-140	34.18	1.8000	Orthogonal	21
S-161	58.79	0.0060	Parallel	12
S-162	58.49	0.0060	Parallel	16
S-163	59.24	0.0060	Parallel	12
S-164	58.90	0.0060	Parallel	12
S-165	58.12	0.0060	Parallel	16
S-166	57.97	0.0060	Parallel	16

S-167	59.10	0.0060	Parallel	21
S-168	58.48	0.0060	Parallel	21
S-169	58.79	0.0060	Parallel	21
S-170	58.83	0.0060	Parallel	21
S-231	58.30	0.0060	Parallel	12
S-232	57.51	0.0060	Parallel	16
S-233	57.51	0.0060	Parallel	12
S-234	59.06	0.0060	Parallel	12
S-235	59.05	0.0060	Parallel	16
S-236	58.33	0.0060	Parallel	16
S-237	57.92	0.0060	Parallel	21
S-238	55.14	0.0060	Parallel	21
S-239	57.05	0.0060	Parallel	21
S-240	56.07	0.0060	Parallel	21
S-181	58.55	0.0180	Parallel	12
S-182	55.42	0.0180	Parallel	16
S-183	58.37	0.0180	Parallel	12
S-184	58.27	0.0180	Parallel	12
S-185	58.34	0.0180	Parallel	16
S-186	58.18	0.0180	Parallel	16
S-187	59.09	0.0180	Parallel	21
S-188	60.47	0.0180	Parallel	21
S-189	57.23	0.0180	Parallel	21
S-190	58.14	0.0180	Parallel	21
S-251	57.37	0.0180	Parallel	12
S-252	54.85	0.0180	Parallel	16
S-253	58.04	0.0180	Parallel	12
S-254	55.00	0.0180	Parallel	12
S-255	55.26	0.0180	Parallel	16
S-256	56.31	0.0180	Parallel	16
S-257	53.94	0.0180	Parallel	21
S-258	53.64	0.0180	Parallel	21
S-259	55.49	0.0180	Parallel	21
S-260	53.78	0.0180	Parallel	21
S-201	55.01	1.8000	Parallel	12
S-202	54.86	1.8000	Parallel	12
S-203	55.11	1.8000	Parallel	12
S-204	51.84	1.8000	Parallel	16
S-205	52.41	1.8000	Parallel	16
S-206	49.66	1.8000	Parallel	16
S-207	47.15	1.8000	Parallel	21

S-208	46.48	1.8000	Parallel	21
S-209	51.96	1.8000	Parallel	21
S-210	51.99	1.8000	Parallel	21
S-271	54.33	1.8000	Parallel	12
S-272	52.84	1.8000	Parallel	16
S-273	55.16	1.8000	Parallel	12
S-274	55.69	1.8000	Parallel	12
S-275	53.26	1.8000	Parallel	16
S-276	48.78	1.8000	Parallel	16
S-277	52.19	1.8000	Parallel	21
S-278	49.17	1.8000	Parallel	21
S-279	50.54	1.8000	Parallel	21
S-280	46.04	1.8000	Parallel	21

Table A.3. Tension Capacity Data from Unstressed Wire Samples Exposed to WD Conditions.

Sample Number	Tension Capacity (kips)	Test Times, t (months)	Observed Temperature, T (°F)	Chloride Concentration, (%Cl-)
195	9.06	3	85	0.0001
196	8.98	3	85	0.0001
197	9.09	3	85	0.0001
210	8.64	6	85	0.0001
211	8.92	6	85	0.0001
212	8.66	6	85	0.0001
227	8.60	9	85	0.0001
228	8.69	9	85	0.0001
229	8.71	9	85	0.0001
244	8.68	12	85	0.0001
245	8.46	12	85	0.0001
246	8.54	12	85	0.0001
198	8.87	3	85	0.006
199	8.93	3	85	0.006
200	8.72	3	85	0.006
213	8.24	6	85	0.006
214	8.65	6	85	0.006
215	8.58	6	85	0.006
230	8.28	9	85	0.006
231	8.11	9	85	0.006
232	7.48	9	85	0.006
247	8.14	12	85	0.006
248	7.83	12	85	0.006
249	7.64	12	85	0.006
201	9.01	3	85	0.018
202	8.79	3	85	0.018
203	8.86	3	85	0.018
216	8.72	6	85	0.018
217	8.46	6	85	0.018
218	8.24	6	85	0.018
233	8.17	9	85	0.018
234	8.04	9	85	0.018
235	7.56	9	85	0.018
250	8.20	12	85	0.018
251	7.96	12	85	0.018
252	8.47	12	85	0.018
204	8.65	3	85	0.18

205	8.98	3	85	0.18
206	8.49	3	85	0.18
219	8.47	6	85	0.18
220	8.79	6	85	0.18
221	8.51	6	85	0.18
222	7.93	6	85	0.18
236	8.35	9	85	0.18
237	8.27	9	85	0.18
238	8.25	9	85	0.18
239	8.39	9	85	0.18
253	7.38	12	85	0.18
254	8.09	12	85	0.18
255	7.82	12	85	0.18
207	8.86	3	85	1.8
208	8.84	3	85	1.8
209	8.74	3	85	1.8
223	7.85	6	85	1.8
224	7.78	6	85	1.8
225	8.40	6	85	1.8
226	7.87	6	85	1.8
240	7.98	9	85	1.8
241	8.13	9	85	1.8
242	7.52	9	85	1.8
243	8.13	9	85	1.8
256	7.54	12	85	1.8
257	7.61	12	85	1.8
258	7.07	12	85	1.8

Table A.4. Tension Capacity Data from Unstressed Wire Samples Exposed to CA Conditions.

Tension Capacity (kips)	Test Times, t (months)	Relative Humidity, RH (%)	Observed Temperature, T (°F)	Chloride Concentration in Surface Grout at GAS Interface, (%gCl), Wt % Cement
9.15	9	97	43	1.05683E-05
9.10	9	97	43	1.05683E-05
9.29	9	45	85	1.05683E-05
9.28	9	70	85	1.05683E-05
9.25	9	70	85	1.05683E-05
9.30	9	45	94	1.05683E-05
9.26	9	97	94	1.05683E-05
8.71	9	97	94	1.05683E-05
8.93	9	97	94	1.05683E-05
9.26	9	70	43	0.013738816
9.26	9	70	43	0.013738816
9.29	9	45	85	0.013738816
9.26	9	45	85	0.013738816
9.16	9	97	85	0.013738816
9.10	9	97	85	0.013738816
9.23	9	97	85	0.013738816
9.24	9	70	94	0.013738816
9.29	9	70	94	0.013738816
9.29	9	45	43	0.091944384
9.30	9	45	43	0.091944384
9.12	9	97	43	0.091944384
9.15	9	97	43	0.091944384
9.20	9	70	85	0.091944384
9.21	9	70	85	0.091944384
9.25	9	70	85	0.091944384
9.19	9	70	85	0.091944384
9.28	9	45	94	0.091944384
9.28	9	45	94	0.091944384
8.94	9	97	94	0.091944384
9.23	9	97	94	0.091944384
9.05	9	97	94	0.091944384
9.12	9	97	94	0.091944384
9.18	9	70	43	0.3434704
9.12	9	70	43	0.3434704
9.20	9	45	85	0.3434704
9.22	9	45	85	0.3434704

9.19	9	45	85	0.3434704
9.13	9	45	85	0.3434704
9.08	9	97	85	0.3434704
9.15	9	97	85	0.3434704
8.99	9	97	85	0.3434704
9.03	9	97	85	0.3434704
9.10	9	70	94	0.3434704
9.16	9	70	94	0.3434704
9.12	9	70	94	0.3434704
9.09	9	70	94	0.3434704
9.13	9	45	43	0.78205568
9.13	9	45	43	0.78205568
9.07	9	97	43	0.78205568
9.02	9	97	43	0.78205568
8.97	9	97	43	0.78205568
9.20	9	70	85	0.78205568
9.03	9	70	85	0.78205568
9.17	9	70	85	0.78205568
9.09	9	70	85	0.78205568
9.13	9	45	94	0.78205568
9.08	9	45	94	0.78205568
6.77	9	97	94	0.78205568
6.04	9	97	94	0.78205568
6.65	9	97	94	0.78205568
7.00	9	97	94	0.78205568

** indicates samples corresponding to *bi-extreme* condition

*** indicates samples corresponding to *tri-extreme* condition

APPENDIX B. STRUCTURAL DETAILING OF CONCRETE REACTION FRAMES

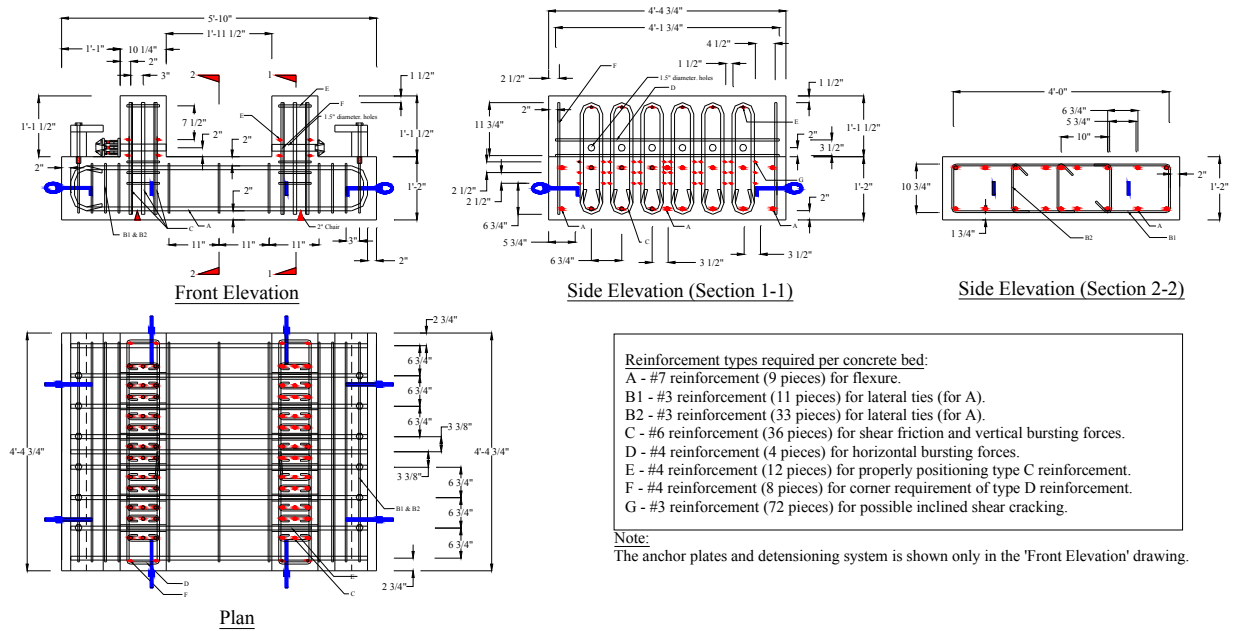


Figure B.1. Structural Detailing of Concrete Reaction Frames – All Views.

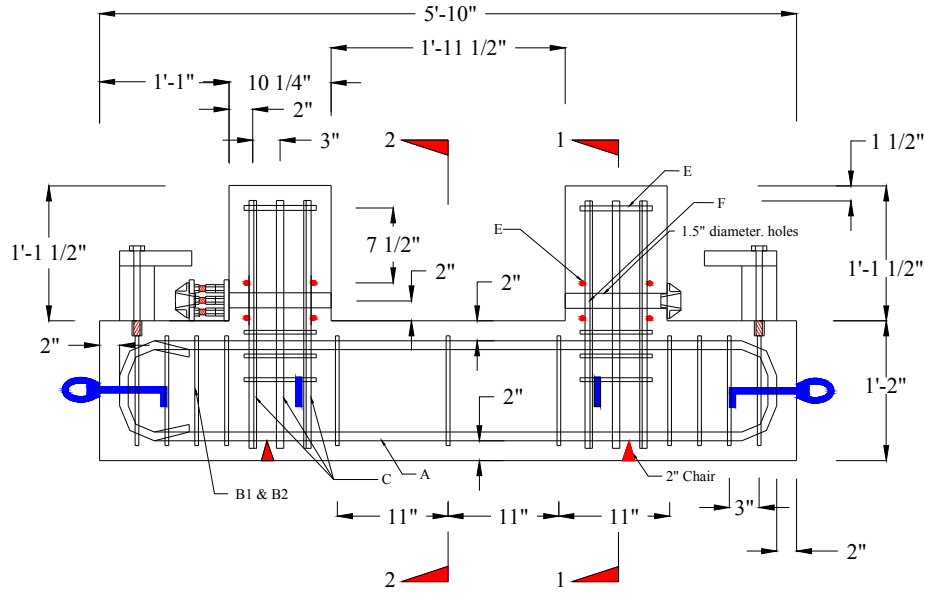


Figure B.2. Structural Detailing of Concrete Reaction Frame – Front Elevation.

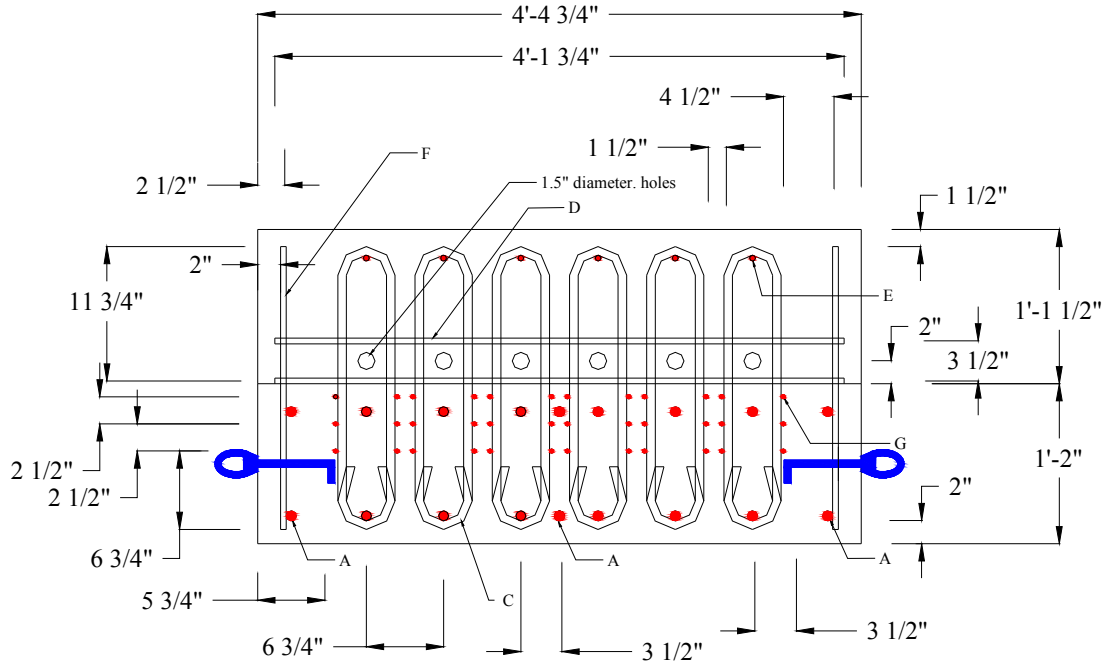


Figure B.3. Structural Detailing of Concrete Reaction Frame – Section 1-1.

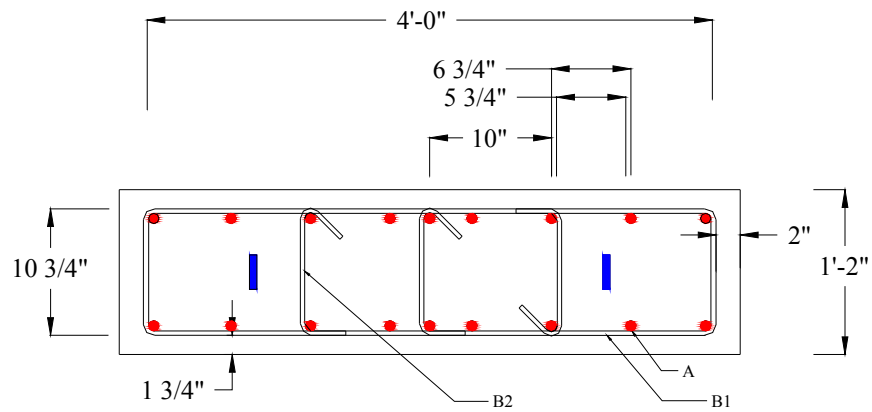


Figure B.4. Structural Detailing of Concrete Reaction Frame – Section 2-2.

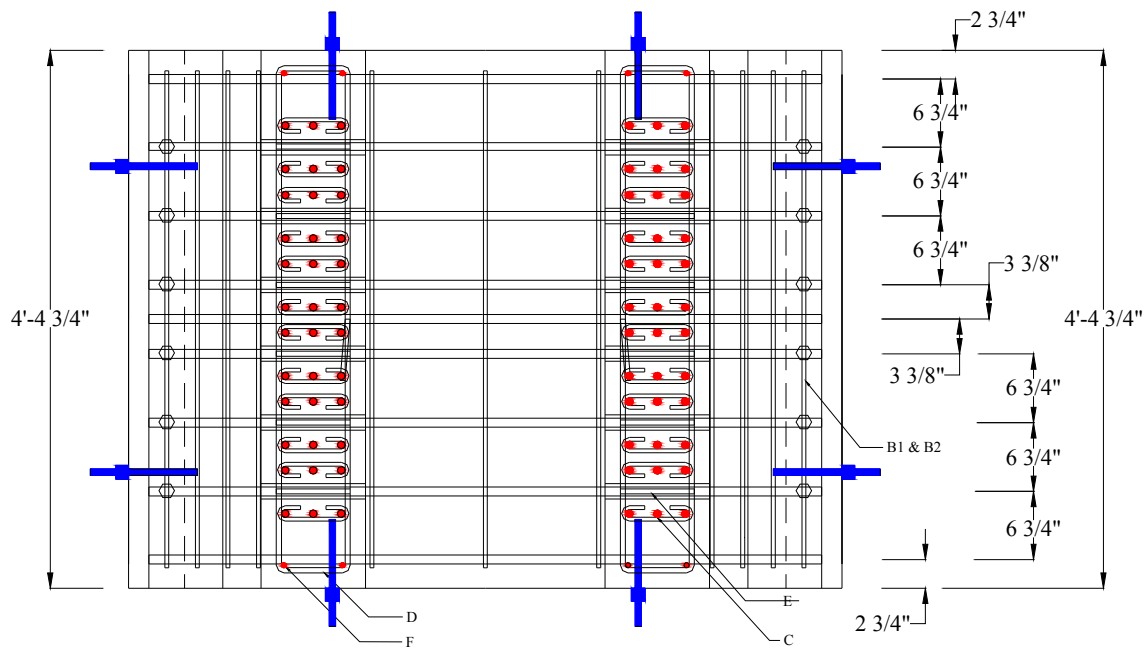
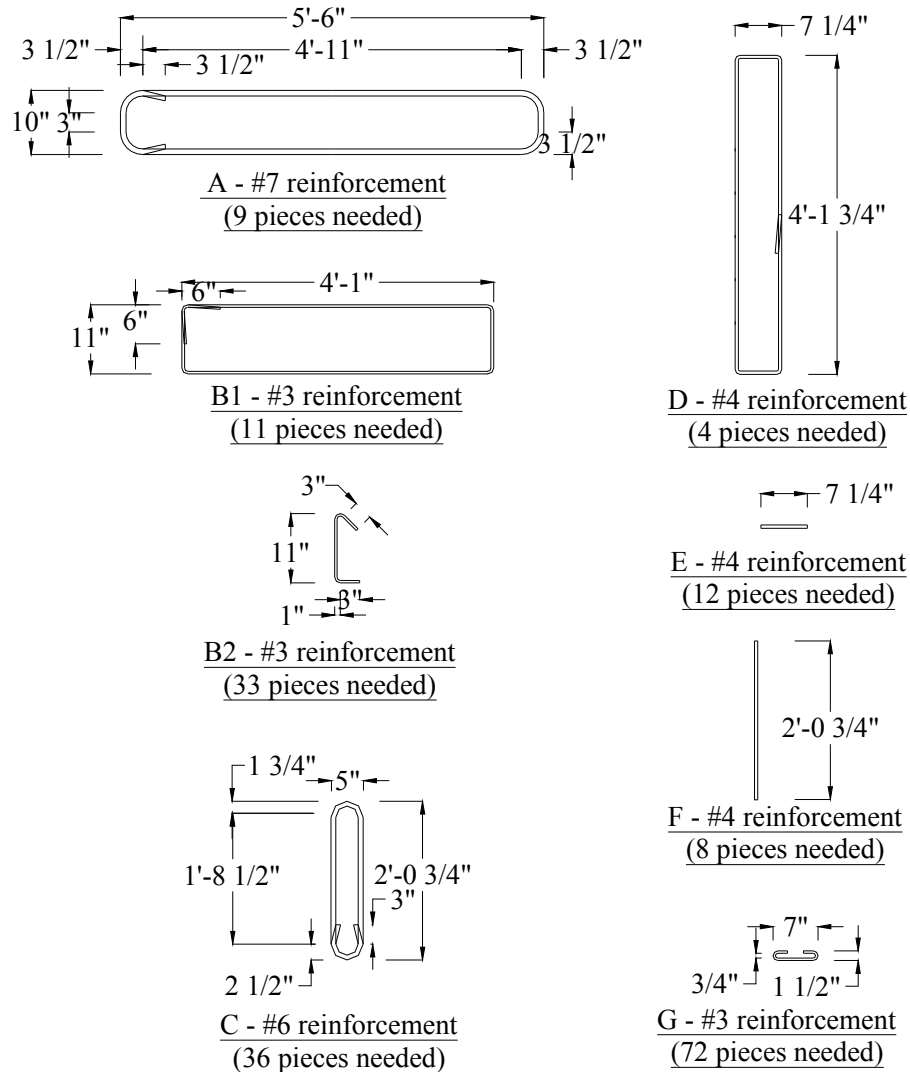


Figure B.5. Structural Detailing of Concrete Reaction Frame – Plan View.



Reinforcement types required per concrete bed:

- A - #7 reinforcement (9 pieces) for flexure.
- B1 - #3 reinforcement (11 pieces) for lateral ties (for A).
- B2 - #3 reinforcement (33 pieces) for lateral ties (for A).
- C - #6 reinforcement (36 pieces) for shear friction and vertical bursting forces.
- D - #4 reinforcement (4 pieces) for horizontal bursting forces.
- E - #4 reinforcement (12 pieces) for properly positioning type C reinforcement.
- F - #4 reinforcement (8 pieces) for corner requirement of type D reinforcement.
- G - #3 reinforcement (72 pieces) for possible inclined shear cracking.

Figure B.6. Detailing of Reinforcement Required for One Concrete Reaction Frame.

APPENDIX C. RESULTS FROM RELIABILITY ASSESSMENT

Table C.1. Data from Strength Reliability Assessment of a Typical PT Bridge.

Sl. No.	Load	%sCI	%gCI	No. of WD tendons	Time (years)	No. of simulations	COV($P_{f,strength}$)	Probability of strength failure, $P_{f,strength}$	Strength reliability index, $\beta_{strength}$
1	HL93	0.006	0.014	0	0			0.0E+00	
2	HL93	0.006	0.014	0	10			0.0E+00	
3	HL93	0.006	0.014	0	20			0.0E+00	
4	HL93	0.006	0.014	0	30			0.0E+00	
5	HL93	0.006	0.014	0	45			0.0E+00	
6	HL93	0.006	0.014	0	75			0.0E+00	
7	HL93	0.006	0.014	1	10			0.0E+00	
8	HL93	0.006	0.014	1	20			0.0E+00	
9	HL93	0.006	0.014	1	30			0.0E+00	
10	HL93	0.006	0.014	1	45			0.0E+00	
11	HL93	0.006	0.014	1	75			0.0E+00	
12	HL93	0.006	0.014	2	10				
13	HL93	0.006	0.014	2	20				
14	HL93	0.006	0.014	2	30	1,900,000	0.0450	2.6E-04	3.47
15	HL93	0.006	0.014	2	45	475,000	0.0520	7.7E-04	3.17
16	HL93	0.006	0.014	2	75	435,502	0.0500	9.2E-04	3.11
17	HL93	0.006	0.014	3	10	6,600,000	0.0788	2.4E-05	4.06
18	HL93	0.006	0.014	3	20	508,000	0.0700	4.0E-04	3.36
19	HL93	0.006	0.014	3	30	508,000	0.0300	1.6E-03	2.96
20	HL93	0.006	0.014	3	45	184,000	0.0400	3.0E-03	2.75
21	HL93	0.006	0.014	3	75	111,468	0.0500	3.6E-03	2.69
22	HL93	0.006	0.014	4	10	5,080,000	0.0496	8.0E-05	3.78
23	HL93	0.006	0.014	4	20	508,000	0.0300	1.4E-03	2.99
24	HL93	0.006	0.014	4	30	92,254	0.0500	4.3E-03	2.63
25	HL93	0.006	0.014	4	45	18,809	0.0500	2.1E-02	2.04
26	HL93	0.006	0.014	4	75	12,506	0.0500	3.1E-02	1.87
27	HL93	0.006	0.014	5	10	1,676,662	0.0500	2.4E-04	3.49
28	HL93	0.006	0.014	5	20	141,515	0.0500	2.8E-03	2.77
29	HL93	0.006	0.014	5	30	17,764	0.0500	2.2E-02	2.01
30	HL93	0.006	0.014	5	45	2,506	0.0500	1.4E-01	1.09
31	HL93	0.006	0.014	5	75	1,573	0.0500	2.0E-01	0.83
32	HL93	0.006	0.014	6	10	718,895	0.0500	5.6E-04	3.26
33	HL93	0.006	0.014	6	20	54,150	0.0500	7.4E-03	2.44
34	HL93	0.006	0.014	6	30	2,929	0.0500	1.2E-01	1.17
35	HL93	0.006	0.014	6	45	577	0.0500	4.1E-01	0.22
36	HL93	0.006	0.014	6	75	411	0.0500	4.9E-01	0.02

37	HL93	0.018	0.092	0	0			0.0E+00	
38	HL93	0.018	0.092	0	10			0.0E+00	
39	HL93	0.018	0.092	0	20			0.0E+00	
40	HL93	0.018	0.092	0	30			0.0E+00	
41	HL93	0.018	0.092	0	45			0.0E+00	
42	HL93	0.018	0.092	0	60			0.0E+00	
43	HL93	0.018	0.092	1	10			0.0E+00	
44	HL93	0.018	0.092	1	20			0.0E+00	
45	HL93	0.018	0.092	1	30			0.0E+00	
46	HL93	0.018	0.092	1	45			0.0E+00	
47	HL93	0.018	0.092	1	60			0.0E+00	
48	HL93	0.018	0.092	2	10				
49	HL93	0.018	0.092	2	20	2,527,240	0.0500	1.6E-04	3.60
50	HL93	0.018	0.092	2	30	755,338	0.0500	5.3E-04	3.27
51	HL93	0.018	0.092	2	45	580,000	0.0430	8.9E-04	3.12
52	HL93	0.018	0.092	2	60	444,456	0.0500	9.0E-04	3.12
53	HL93	0.018	0.092	3	10	2,540,000	0.0764	6.7E-05	3.82
54	HL93	0.018	0.092	3	20	428,284	0.0490	9.3E-04	3.11
55	HL93	0.018	0.092	3	30	720,000	0.0230	2.6E-03	2.80
56	HL93	0.018	0.092	3	45	110,237	0.0500	3.6E-03	2.69
57	HL93	0.018	0.092	3	60	110,867	0.0500	3.6E-03	2.69
58	HL93	0.018	0.092	4	10	1,400,000	0.0550	2.3E-04	3.50
59	HL93	0.018	0.092	4	20	147,678	0.0500	2.7E-03	2.78
60	HL93	0.018	0.092	4	30	483,000	0.0100	1.1E-02	2.28
61	HL93	0.018	0.092	4	45	12,483	0.0500	3.1E-02	1.87
62	HL93	0.018	0.092	4	60	12,480	0.0500	3.1E-02	1.87
63	HL93	0.018	0.092	5	10	544,191	0.0500	7.4E-04	3.18
64	HL93	0.018	0.092	5	20	47,661	0.0500	8.3E-03	2.39
65	HL93	0.018	0.092	5	30	4,059	0.0500	9.0E-02	1.34
66	HL93	0.018	0.092	5	45	1,573	0.0500	2.0E-01	0.83
67	HL93	0.018	0.092	5	60	1,317	0.0500	2.3E-01	0.73
68	HL93	0.018	0.092	6	10	260,347	0.0500	1.5E-03	2.96
69	HL93	0.018	0.092	6	20	9,778	0.0500	3.9E-02	1.76
70	HL93	0.018	0.092	6	30	1,000	0.0500	2.9E-01	0.57
71	HL93	0.018	0.092	6	45	424	0.0500	4.9E-01	0.04
72	HL93	0.018	0.092	6	60	415	0.0500	4.9E-01	0.02
73	HL93	1.8	0.782	0	0			0.0E+00	
74	HL93	1.8	0.782	0	5			0.0E+00	
75	HL93	1.8	0.782	0	10			0.0E+00	
76	HL93	1.8	0.782	0	15			0.0E+00	
77	HL93	1.8	0.782	0	20			0.0E+00	
78	HL93	1.8	0.782	0	30			0.0E+00	

79	HL93	1.8	0.782	1	5			0.0E+00	
80	HL93	1.8	0.782	1	10			0.0E+00	
81	HL93	1.8	0.782	1	15			0.0E+00	
82	HL93	1.8	0.782	1	20			0.0E+00	
83	HL93	1.8	0.782	1	30			0.0E+00	
84	HL93	1.8	0.782	2	5				
85	HL93	1.8	0.782	2	10	2,540,000	0.0513	1.5E-04	3.62
86	HL93	1.8	0.782	2	15	755,359	0.0500	5.3E-04	3.27
87	HL93	1.8	0.782	2	20	456,780	0.0500	8.8E-04	3.13
88	HL93	1.8	0.782	2	30	387,870	0.0500	1.0E-03	3.08
89	HL93	1.8	0.782	3	5	2,540,000	0.0770	6.6E-05	3.82
90	HL93	1.8	0.782	3	10	406,054	0.0500	9.9E-04	3.09
91	HL93	1.8	0.782	3	15	141,094	0.0500	2.8E-03	2.77
92	HL93	1.8	0.782	3	20	93,045	0.0500	4.3E-03	2.63
93	HL93	1.8	0.782	3	30	81,609	0.0500	4.9E-03	2.58
94	HL93	1.8	0.782	4	5	552,665	0.0500	7.2E-04	3.19
95	HL93	1.8	0.782	4	10	131,793	0.0500	3.0E-03	2.74
96	HL93	1.8	0.782	4	15	35,150	0.0500	1.1E-02	2.28
97	HL93	1.8	0.782	4	20	14,072	0.0500	2.8E-02	1.92
98	HL93	1.8	0.782	4	30	10,462	0.0500	3.7E-02	1.79
99	HL93	1.8	0.782	5	5	563,550	0.0500	7.1E-04	3.19
100	HL93	1.8	0.782	5	10	46,974	0.0500	8.5E-03	2.39
101	HL93	1.8	0.782	5	15	4,300	0.0500	8.5E-02	1.37
102	HL93	1.8	0.782	5	20	1,502	0.0500	2.1E-01	0.81
103	HL93	1.8	0.782	5	30	879	0.0500	3.1E-01	0.49
104	HL93	1.8	0.782	6	5	266,124	0.0500	1.5E-03	2.96
105	HL93	1.8	0.782	6	10	10,261	0.0500	3.8E-02	1.78
106	HL93	1.8	0.782	6	15	679	0.0500	3.7E-01	0.33
107	HL93	1.8	0.782	6	20	231	0.0500	6.4E-01	-0.35
108	HL93	1.8	0.782	6	30	149	0.0500	7.3E-01	-0.62
109	HS20	0.006	0.014	0	0			0.0E+00	
110	HS20	0.006	0.014	0	10			0.0E+00	
111	HS20	0.006	0.014	0	20			0.0E+00	
112	HS20	0.006	0.014	0	30			0.0E+00	
113	HS20	0.006	0.014	0	45			0.0E+00	
114	HS20	0.006	0.014	0	75			0.0E+00	
115	HS20	0.006	0.014	1	10			0.0E+00	
116	HS20	0.006	0.014	1	20			0.0E+00	
117	HS20	0.006	0.014	1	30			0.0E+00	
118	HS20	0.006	0.014	1	45			0.0E+00	
119	HS20	0.006	0.014	1	75			0.0E+00	
120	HS20	0.006	0.014	2	10				

121	HS20	0.006	0.014	2	20				
122	HS20	0.006	0.014	2	30				
123	HS20	0.006	0.014	2	45				
124	HS20	0.006	0.014	2	75	580,000	0.0900	2.1E-04	3.53
125	HS20	0.006	0.014	3	10				
126	HS20	0.006	0.014	3	20	3,200,000	0.0700	6.3E-05	3.84
127	HS20	0.006	0.014	3	30	664,399	0.0500	6.0E-04	3.24
128	HS20	0.006	0.014	3	45	197,603	0.0500	2.0E-03	2.87
129	HS20	0.006	0.014	3	75	165,387	0.0500	2.4E-03	2.82
130	HS20	0.006	0.014	4	10	5,400,000	0.1690	6.5E-06	4.36
131	HS20	0.006	0.014	4	20	839,589	0.0500	4.8E-04	3.30
132	HS20	0.006	0.014	4	30	161,237	0.0500	2.5E-03	2.81
133	HS20	0.006	0.014	4	45	65,708	0.0500	6.1E-03	2.51
134	HS20	0.006	0.014	4	75	50,648	0.0500	7.8E-03	2.42
135	HS20	0.006	0.014	5	10	3,500,000	0.1190	2.8E-05	4.03
136	HS20	0.006	0.014	5	20	237,199	0.0500	1.7E-03	2.93
137	HS20	0.006	0.014	5	30	60,061	0.0500	6.6E-03	2.48
138	HS20	0.006	0.014	5	45	11,756	0.0500	3.3E-02	1.84
139	HS20	0.006	0.014	5	75	6,409	0.0500	5.9E-02	1.56
140	HS20	0.006	0.014	6	10	4,506,628	0.0500	8.9E-05	3.75
141	HS20	0.006	0.014	6	20	106,172	0.0500	3.8E-03	2.67
142	HS20	0.006	0.014	6	30	15,747	0.0500	2.5E-02	1.96
143	HS20	0.006	0.014	6	45	1,688	0.0500	1.9E-01	0.87
144	HS20	0.006	0.014	6	75	1,279	0.0500	2.4E-01	0.71
145	HS20	0.018	0.092	0	0			0.0E+00	
146	HS20	0.018	0.092	0	10			0.0E+00	
147	HS20	0.018	0.092	0	20			0.0E+00	
148	HS20	0.018	0.092	0	30			0.0E+00	
149	HS20	0.018	0.092	0	45			0.0E+00	
150	HS20	0.018	0.092	0	60			0.0E+00	
151	HS20	0.018	0.092	1	10			0.0E+00	
152	HS20	0.018	0.092	1	20			0.0E+00	
153	HS20	0.018	0.092	1	30			0.0E+00	
154	HS20	0.018	0.092	1	45			0.0E+00	
155	HS20	0.018	0.092	1	60			0.0E+00	
156	HS20	0.018	0.092	2	10				
157	HS20	0.018	0.092	2	20				
158	HS20	0.018	0.092	2	30				
159	HS20	0.018	0.092	2	45				
160	HS20	0.018	0.092	2	60	355,000	0.1100	2.3E-04	3.51
161	HS20	0.018	0.092	3	10	2,800,000	0.2600	5.4E-07	4.88
162	HS20	0.018	0.092	3	20	508,000	0.0820	2.9E-04	3.44

163	HS20	0.018	0.092	3	30	292,589	0.0500	1.4E-03	3.00
164	HS20	0.018	0.092	3	45	158,289	0.0500	2.5E-03	2.80
165	HS20	0.018	0.092	3	60	165,686	0.0500	2.4E-03	2.82
166	HS20	0.018	0.092	4	10	5,000,000	0.0840	2.8E-05	4.03
167	HS20	0.018	0.092	4	20	254,559	0.0500	1.6E-03	2.95
168	HS20	0.018	0.092	4	30	85,669	0.0500	4.7E-03	2.60
169	HS20	0.018	0.092	4	45	51,219	0.0500	7.8E-03	2.42
170	HS20	0.018	0.092	4	60	53,427	0.0500	7.5E-03	2.43
171	HS20	0.018	0.092	5	10	840,000	0.0870	1.6E-04	3.61
172	HS20	0.018	0.092	5	20	102,536	0.0500	3.9E-03	2.66
173	HS20	0.018	0.092	5	30	23,056	0.0500	1.7E-02	2.11
174	HS20	0.018	0.092	5	45	7,115	0.0500	5.3E-02	1.61
175	HS20	0.018	0.092	5	60	6,517	0.0500	5.8E-02	1.57
176	HS20	0.018	0.092	6	10	600,000	0.0560	5.2E-04	3.28
177	HS20	0.018	0.092	6	20	41,220	0.0500	9.6E-03	2.34
178	HS20	0.018	0.092	6	30	3,051	0.0500	1.2E-01	1.20
179	HS20	0.018	0.092	6	45	1,093	0.0500	2.7E-01	0.62
180	HS20	0.018	0.092	6	60	1,150	0.0500	2.6E-01	0.65
181	HS20	1.8	0.782	0	0			0.0E+00	
182	HS20	1.8	0.782	0	5			0.0E+00	
183	HS20	1.8	0.782	0	10			0.0E+00	
184	HS20	1.8	0.782	0	15			0.0E+00	
185	HS20	1.8	0.782	0	20			0.0E+00	
186	HS20	1.8	0.782	0	30			0.0E+00	
187	HS20	1.8	0.782	1	5			0.0E+00	
188	HS20	1.8	0.782	1	10			0.0E+00	
189	HS20	1.8	0.782	1	15			0.0E+00	
190	HS20	1.8	0.782	1	20			0.0E+00	
191	HS20	1.8	0.782	1	30			0.0E+00	
192	HS20	1.8	0.782	2	5				
193	HS20	1.8	0.782	2	10				
194	HS20	1.8	0.782	2	15				
195	HS20	1.8	0.782	2	20				
196	HS20	1.8	0.782	2	30	350,000	0.1000	2.6E-04	3.47
197	HS20	1.8	0.782	3	5				
198	HS20	1.8	0.782	3	10	1,646,890	0.0500	2.4E-04	3.49
199	HS20	1.8	0.782	3	15	278,035	0.0500	1.4E-03	2.98
200	HS20	1.8	0.782	3	20	168,760	0.0500	2.4E-03	2.82
201	HS20	1.8	0.782	3	30	135,489	0.0500	2.9E-03	2.75
202	HS20	1.8	0.782	4	5	3,600,000	0.1021	2.7E-05	4.04
203	HS20	1.8	0.782	4	10	250,880	0.0500	1.6E-03	2.95
204	HS20	1.8	0.782	4	15	75,099	0.0500	5.3E-03	2.56

205	HS20	1.8	0.782	4	20	51,267	0.0500	7.7E-03	2.42
206	HS20	1.8	0.782	4	30	41,788	0.0500	9.5E-03	2.35
207	HS20	1.8	0.782	5	5	2,475,000	0.0554	1.3E-04	3.65
208	HS20	1.8	0.782	5	10	88,918	0.0500	4.5E-03	2.61
209	HS20	1.8	0.782	5	15	22,998	0.0500	1.7E-02	2.12
210	HS20	1.8	0.782	5	20	7,877	0.0500	4.8E-02	1.66
211	HS20	1.8	0.782	5	30	4,910	0.0500	7.5E-02	1.44
212	HS20	1.8	0.782	6	5	600,000	0.0580	4.9E-04	3.30
213	HS20	1.8	0.782	6	10	40,962	0.0500	9.7E-03	2.34
214	HS20	1.8	0.782	6	15	3,205	0.0500	1.1E-01	1.22
215	HS20	1.8	0.782	6	20	887	0.0500	3.1E-01	0.49
216	HS20	1.8	0.782	6	30	569	0.0500	4.1E-01	0.22