Technical Report Documentation Page

1. Report No. FHWA/TX-04/0-1405-9	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Conclusions, Recommendations and Design Guidelines for Corrosion Protection of Post-Tensioned Bridges		5. Report Date February 2004 6. Performing Organization Code
7. Author(s) R. M. Salas, A. J. Schokker, J. S. West, J. E. Breen, and M. E. Kreger		8. Performing Organization Report No. Research Report 0-1405-9
9. Performing Organization Name and Address Center for Transportation Research, The University of Texas at Austin, 3208 Red River, Suite 200, Austin, TX 78705-2650		10. Work Unit No. (TRAIS) 11. Contract or Grant No. Research Study 0-1405
12. Sponsoring Agency Name and Address Texas Department of Transportation, Research and Technology Implementation Office, P.O. Box 5080, Austin, TX 78763-5080		13. Type of Report and Period Covered Research Report (9/93-8/03) 14. Sponsoring Agency Code

15. Supplementary Notes

Project conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration, and the Texas Department of Transportation.

16. Abstract

The effectiveness of cement grout in galvanized or polyethylene ducts, the most widely used corrosion protection system for multistrand bonded post-tensioned concrete tendons, has been under debate due to several reported examples of significant tendon corrosion damage. While experience in the USA has been generally good, some foreign experience has been less than satisfactory.

This report is the last technical report from a comprehensive research program started in 1993 under TxDOT Project 0-1405. The objectives were to examine the use of post-tensioning in bridge substructures, identify durability concerns and existing technology, develop and carry out an experimental testing program, and conclude with durability design guidelines.

Four experimental programs were developed: improved and high-performance grout studies, to develop grout with desirable fresh properties to provide good corrosion protection to the prestressing strands; a long-term macrocell corrosion test series, to investigate corrosion protection for internal tendons in precast segmental construction; a long-term beam corrosion test series, to examine the effects of post-tensioning on corrosion protection as affected by crack width; and, a long-term column corrosion test series, to examine corrosion protection in vertical elements.

This report includes the final results after completion of exposure testing, performing comprehensive autopsies and updating the durability design guidelines to reduce the corrosion risk of the post-tensioning system.

After autopsies were performed, overall findings indicate negative durability effects due to the use of mixed reinforcement, small concrete covers, galvanized steel ducts, and industry standard or heat-shrink galvanized duct splices. The width of cracks was shown to have a direct negative effect on specimen performance. Grout voids were found to be detrimental to the durability of both galvanized ducts and strand. Relying on epoxy and galvanized bar coatings was also found inappropriate because of local attack. On the other hand, very positive effects were found with the use of high performance concrete, high-performance grouts, high post-tensioning levels, plastic ducts, and sound epoxy filling at the joints.

17. Key Words	18. Distribution Statement
post-tensioned concrete, tendons, corrosion protection, grouts, ducts, high-performance concrete, epoxy joints, reinforcement coatings	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	100	

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CONCLUSIONS, RECOMMENDATIONS AND DESIGN GUIDELINES FOR CORROSION PROTECTION OF POST-TENSIONED BRIDGES

by

R. M. Salas, A. J. Schokker, J. S. West,
J. E. Breen, and M. E. Kreger

Research Report 0-1405-9

Research Project 0-1405

DURABILITY DESIGN OF POST-TENSIONED
BRIDGE SUBSTRUCTURE ELEMENTS

conducted for the
Texas Department of Transportation

in cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

February 2004

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

ACKNOWLEDGMENTS

We greatly appreciate the financial support from the Texas Department of Transportation that made this project possible. We are grateful for the active support of the project director, Bryan Hodges (TYL), and the support of program coordinator, Richard Wilkison is also very much appreciated. We thank Project Monitoring Committee members, Gerald Lankes (CST), Ronnie VanPelt (BMT), and Tamer Ahmed (FHWA). We also are deeply grateful for the many contributions of a dedicated and talented group of graduate students who participated in earlier phases of this study including: Rene P. Vignos, Andrea L. Kotys, Bradley D. Koester, Carl J. Larosche and H.R. Hamilton. Finally, we would like to thank Dr. Harovel Wheat for her invaluable guidance throughout the project.

DISCLAIMER

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Research Supervisors

TABLE OF CONTENTS

CHAPTER 1: INTRODUCTION..1

1.1 Background..1

1.2 Designing Bonded Post-Tensioned Systems for Durability 3

1.3 Research Objectives and Scope 4

1.3.1 Project Statement 4

1.3.2 Project Objectives..5

1.3.3 Project Scope 5

1.3.4 Project Reports...7

CHAPTER 2: INTERNAL BONDED POST-TENSIONING SYSTEMS..11

2.1 Prestressed Concrete 11

2.1.1 Historical Development 11

2.1.2 Basic Definitions...12

2.1.3 Advantages and Disadvantages of Internal and External Post-Tensioning 13

2.1.4 Mixed Post-Tensioning...14

2.1.5 Applications of Bonded Post-Tensioning 15

2.1.6 Mixed Reinforcement 18

2.2 Durability of Bonded Post-Tensioned Concrete Structures.............................18

2.2.1 Problems Encountered Around the World: The Ghent Workshop 19

2.2.2 Experience in United States...20

2.3 Factors Affecting The Durability of Bonded Post-Tensioned Concrete Structures 24

2.3.1 Exposure Conditions..24

2.3.2 Concrete Durability..26

2.3.3 Grouts for Post-Tensioning..27

2.3.4 Corrosion of Steel Components in Post-Tensioning Systems 28

2.3.5 Anchorage Sealing...33

2.3.6 Precast Segmental Construction – Joint Performance 33

2.4 Corrosion Protection of Bonded Post-Tensioned Concrete Structures.................34

2.4.1 Structural Form 35

2.4.2 Structural Design Details 36
2.4.3 Surface Treatments 36
2.4.4 Concrete as Corrosion Protection 36
2.4.5 High Performance Grouts 37
2.4.6 Ducts for Post-Tensioning 38
2.4.7 Coatings on Prestressing Steel 38
2.4.8 Prestressing Steel – Other Prestressing Materials 39
2.4.9 Anchorage Protection 39
2.4.10 Encapsulated and Electrically Isolated Systems 40
2.4.11 Precast Segmental Construction – Joint Detail 41
2.4.12 Temporary Corrosion Protection 41
2.4.13 Inspection Practices 41
2.4.14 Monitoring 41

CHAPTER 3: PROJECT SUMMARY AND CONCLUSIONS 43

3.1 Summary and Conclusions from Improved and High-Performance Grout Studies 43
3.1.1 Fresh Property Tests 43
3.1.2 Accelerated Corrosion Tests 44
3.1.3 Large-Scale Duct Tests 45

3.2 Summary and Conclusions from Macrocell Corrosion Tests 46
3.2.1 Overall Performance 48
3.2.2 Assessing Corrosion Activity Using Half-Cell Potential Measurements 48
3.2.3 Segmental Joints 48
3.2.4 Ducts for Internal Post-Tensioning 49
3.2.5 Joint Precompression 49
3.2.6 Grouts for Bonded Post-Tensioning 50

3.3 Summary and Conclusions from Beam Corrosion Tests 50
3.3.1 Overall Performance 52
3.3.2 Load/Prestress Level versus Corrosion: The Effect of Cracking 52
3.3.3 Fly Ash in Concrete 53
3.3.4 Duct Splices for Galvanized Steel Duct 53
3.3.5 High Performance Fly Ash Grouts 53
3.3.6 Exposure Testing Results 54

3.4 Summary and Conclusions from Column Corrosion Tests 54
3.4.1 Post-Tensioning to Improve Corrosion Protection 56
3.4.2 Fly Ash as Partial Cement Replacement in Concrete 57
3.4.3 Plastic Ducts for Post-Tensioning....57
3.4.4 Post-Tensioning Bar Coatings 57

3.5 Major Overall Conclusions....57

3.6 Recommended Changes and Procedures 63

3.7 Directions for Future Research....63

CHAPTER 4: IMPLEMENTATION RECOMMENDATIONS....65

4.1 Measures For Implementation From Improved and High-Performance Grout Studies 65

4.2 Measures For Implementation From Macrocell Corrosion Tests....65

4.3 Measures For Implementation From Beam Corrosion Tests....66

4.4 Measures For Implementation From Column Corrosion Tests 67

REFERENCES....69

LIST OF TABLES

Table 1.1 Major Project Tasks and Contributions of Graduate Students..................................... 7
Table 1.2 Proposed Project 0-1405 Reports.. 9
Table 1.3 Project 0-1405 Theses and Dissertations, The University of Texas at Austin.............. 9
Table 2.1 Key Developments in Prestressed Concrete History[42].. 11
Table 2.2 Basic Definitions... 13
Table 2.3 Advantages and Disadvantages of Internal and External Post-Tensioning 14
Table 2.4 Common Reference Electrode Potentials versus SHE[60] 30

LIST OF FIGURES

Figure 1.1 Simplified Analogy between Design for Structural Loading and Durability 4
Figure 1.2 TxDOT Project 0-1405 Scope, Researchers and Technical Reports 6
Figure 2.1 Post-Tensioning Anchorage Details[34] 12
Figure 2.2 Typical Balanced Cantilever Segment[16] 15
Figure 2.3 Vertical Post-Tensioning of the High-Level Approach Piers of the Sunshine Skyway Bridge, Florida[16] 16
Figure 2.4 Post-Tensioning in (a) Hammerhead Piers, (b) Cantilever Piers, and (c) Straddle Bents[16] 17
Figure 2.5 Plan View of Slipped Tendon at Deviation Saddle Niles Channel Bridge[16] 20
Figure 2.6 Advanced Corrosion of Strands within Anchorage Niles Channel Bridge[16] 21
Figure 2.7 The Mid-Bay Bridge, Florida[16] 21
Figure 2.8 Failure of Tendon 28-6 on the Mid-Bay Bridge[16] 22
Figure 2.9 Failure of Tendon 57-1 on the Mid-Bay Bridge at Expansion Joint Diaphragm[16] ...22
Figure 2.10 The Sunshine Skyway Bridge, Tampa, Florida [16] 23
Figure 2.11 Tendon Corrosion inside the Sunshine Skyway Bridge Piers[16] 23
Figure 2.12 Passive-Active Behavior in Steel[34] 29
Figure 2.13 Corrosion of Steel in Concrete[34] 30
Figure 2.14 The Self-Propagating Process of Pitting Corrosion[34] 32
Figure 2.15 Multilayer Corrosion Protection for Bonded Post-Tensioned Systems[33] 34
Figure 2.16 Hazard Scenarios[68] 35
Figure 2.17 Multi-Layer Corrosion Protection for Buried Post-Tensioning Anchorages[22] 40
Figure 2.18 Member End Details for Anchorage Corrosion Protection[22] 40
Figure 3.1 Large-Scale Clear Draped Parabolic Duct Test[34] 43
Figure 3.2 Schematic of ACTM Station[34] 44
Figure 3.3 ACTM Test Setup[34] 44
Figure 3.4 Comparison of ACTM Average Times to Corrosion[34] 45
Figure 3.5 TxDOT Grout – Bubbles Traveling Toward Intermediate Crest[34] 45
Figure 3.6 Beginning of Void Formation at Intermediate Crest[34] 46
Figure 3.7 Comparison of Slices from the Intermediate Crest[34] 46
Figure 3.8 Macrocell Test Setup[83] 47
Figure 3.9 Macrocell specimen details[83] 47
Figure 3.10 Example of Specimen Condition Prior to Autopsy Specimen DJ-S-L-NG-2[83] 47
Figure 3.11 Beam Test Setup at North End of Ferguson Laboratory[37] 51
Figure 3.12 Test Setup[33] 51
Figure 3.13 Example of Specimen Condition Prior to Autopsy Specimen 2.3 - Mixed Reinforced Beam under Constant Service Load[37,38] 51
Figure 3.14 Column Test Setup [37] 55
Figure 3.15 Column-Foundation Joint Configurations[37] 55
Figure 3.16 Column concrete Removal and Reinforcement Dismantling[37] 56
Figure 3.17 Large Galvanized Steel Duct Destruction Due to Corrosion[37,38] 58
Figure 3.18 Effect of Transverse Concrete Cracking Due to Service Loading on Reinforcement Corrosion[37,38] 59
Figure 3.19 Example of Longitudinal (Splitting) Crack in Macrocell Specimen[83] 59
Figure 3.20 Segmental or Construction Joint Performance[37] 60

Figure 3.21 Corrosion in Galvanized Steel Duct in Beams with Different Levels of Post-Tensioning[37,38] 60
Figure 3.22 Duct Splice Performance[37] 61
Figure 3.23 Effect of the Use of Rubber Gaskets[37] 61
Figure 3.24 Bleed Water Voids and Duct Corrosion[37] 62
Figure 3.25 Effect of Grout Cracking[37] 62
Figure 3.26 Bar Coating Performance when Subjected to Localized Attack[37] 63

SUMMARY

The effectiveness of cement grout in galvanized or polyethylene ducts, the most widely used corrosion protection system for multistrand bonded post-tensioned concrete tendons, has been under debate, due to significant tendon corrosion damage, several reported failures of individual tendons as well as a few collapses of non-typical structures. While experience in the USA has been generally good, some foreign experience has been less than satisfactory.

This report is the last technical report from a comprehensive research program started in 1993 under TxDOT Project 0-1405. The objectives were to examine the use of post-tensioning in bridge substructures, identify durability concerns and existing technology, develop and carry out an experimental testing program, and conclude with durability design guidelines.

Four experimental programs were developed: Improved and high-performance grout studies, to develop grout with desirable fresh properties to provide good corrosion protection to the prestressing strands; a long term macrocell corrosion test series, to investigate corrosion protection for internal tendons in precast segmental construction; a long term beam corrosion test series, to examine the effects of post-tensioning on corrosion protection as affected by crack width; and, a long term column corrosion test series, to examine corrosion protection in vertical elements.

Preliminary design guidelines were developed previously and were published in Reports 1405-2 and 1405-5, after an extensive literature review and the preliminary results from specimen exposure testing. Guidance was provided for assessing the durability risk and for ensuring protection against freeze-thaw damage, sulfate attack and corrosion of steel reinforcement.

This report includes the final results after continuation of exposure testing of the macrocell, beam and column specimens, performing comprehensive autopsies of selected specimens and updating the durability design guidelines based on the exposure testing and autopsy results. In this report the emphasis is on the corrosion risk of the post-tensioning system.

After autopsies were performed, overall findings indicate negative durability effects due to the use of mixed reinforcement, small concrete covers, galvanized steel ducts, and industry standard or heat-shrink galvanized duct splices. The width of cracks was shown to have a direct negative effect on specimen performance. Grout voids were found to be detrimental to the durability of both galvanized ducts and strand. Relying on epoxy and galvanized bar coatings was also found inappropriate because of local attack. On the other hand, very positive effects were found with the use of high performance concrete, high-performance grouts, high post-tensioning levels, plastic ducts, and sound epoxy filling at the joints.

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

In the last few years, the effectiveness of cement grout in galvanized or polyethylene ducts, the most widely used corrosion protection system for internal and external multistrand post-tensioning for bridge superstructures has been under debate, due to reports of a few instances of significant tendon corrosion damage, several reported failures of individual tendons as well as a few collapses of very non-typical structures.[1-5] While experience in the USA has been generally quite good,[6] some foreign experience has been less than satisfactory. A moratorium was established in the U.K. in 1992 for internal bonded post-tensioned structures (due to bridge failures including the well known collapse of the Ynys-y-Gwas Bridge in Wales), and is still in effect for "internal, grouted tendons with discontinuous (poorly sealed) ducts."[2,7] Germany has expressed a preference for the use of external prestressing.[8-9] The French Authorities have gone in the opposite direction considering the idea of forbidding external tendons injected with cement grouts.[10] Japan has expressed a preference for the use of fully external tendons using transparent sheath with grouting.[4] These are only a few examples of the general concern and show the need for comprehensive studies regarding the corrosion protection of bonded post-tensioning systems.

Recognizing the extent of the problem, in November 2001, engineers from many countries gathered at Ghent University,[11] Belgium, under the sponsorship of *fib* (federation internationale du béton) and IABSE (International Association for Bridge and Structural Engineering), to review the problems encountered and to discuss the available solutions. Other congresses and seminars have followed, including the October 2002, first *fib* Congress "Concrete Structures in the 21st Century," in Osaka, Japan. However, many aspects still remain under discussion.

In United States, the very limited problems with tendon corrosion in precast segmental bridges[6,12-17] include one external tendon failure found in 1999 in the Niles Channel Bridge, two tendon failures and eleven corroded tendons discovered during the year 2000 at the Mid-Bay Bridge, and, corroded and failed vertical tendons discovered in the same year in precast segmental columns of the high level approaches of the Sunshine Skyway Bridge. All of these bridges are located in the State of Florida. Additionally, grouting deficiencies were found in 2001 in the Sidney Lanier cable-stayed bridge in Georgia and in the Boston Central Artery bridges. No significant tendon corrosion problems have been reported in the states of California (with 3800 post-tensioned bridges), Georgia, Texas and Virginia, as reported by the American Segmental Bridge Association.[18]

The general concern after the unfortunate experiences world wide has lead many transportation agencies and technical societies to produce "emergency" documents and technical reports.[19,20,21] These state of the art reports and specification documents are the response to knowledge and expertise to date, with regard to new material requirements, construction practices and monitoring techniques. However, it is clear, that more research is needed to reinforce or even to disprove some of these theories, since to date there is not a clear agreement on a consistent set of durability design, construction and monitoring guidelines. Some of the mentioned "emergency" documents include:

- **The Concrete Society (1996),** "Durable Bonded Post-Tensioned Concrete Bridges," The Concrete Society Technical Report TR47, United Kingdom, 1996.[22] Second edition to be published shortly as reported by Prof. G. Somerville.[23]
- **FIP (1996),**"Corrosion protection of Prestressing Steels," Fédération Internationale de la Précontrainte (FIP), London, 1996.[24]

- ***fib* (2000)**, "Corrugated Plastic Ducts for Internal Bonded Post-Tensioning," Fédération Internationale du béton (*fib*) Technical Report Bulletin No. 7, Lausanne, Switzerland, January, 2000.[25]
- ***fib* (2000),** "Grouting of tendons in prestressed concrete," Fédération Internationale du béton (*fib*) Technical Report Bulletin No. 20, Lausanne, Switzerland, July, 2000.[26]
- **JPCEA (2001),** "Manual for Maintenance of Prestressed Concrete Bridges," Japan, November 2000. As referenced in Hamada[1.27] and Kitazono.[28]
- **ASBI (2001),** "Interim Statement on Grouting Practices," Phoenix, Arizona, U.S.A., December 2000.[20]
- **JSCE (2001),** "Standard Specification for Maintenance of Concrete Structures," Japan, January 2001. As referenced by Hamada[1.27] and Kitazono.[28]
- **PTI (2001),** "Guide Specification for Grouting of Post-Tensioned Structures," Phoenix, Arizona, U.S.A. February 2001, First edition.[19]
- **ASBI (2002),** "2002 Grouting Certification Training Manual," Phoenix, Arizona, U.S.A., 2002.[21]
- **FLDOT (2002),** "New Directions for Florida Post-Tensioned Bridges," Tallahassee, Florida, U.S.A., February 2002.[16]
- **VSL (2002),** "Grouting of Post-Tensioning Tendons," VSL International Ltd. Lyssach, Switzerland, May 2002.[23]
- **Swiss Federal Roads Authority and the Swiss Federal Railways (2001)**, "Measures for providing durability of post-tensioning tendons in bridges," [29]

Corrosion protection for bonded internal tendons can be very effective. Within the elements, internal tendons can be well protected by the multilayer protection system; including a sound design taking away the surface water, surface treatments, high quality concrete, plastic or galvanized duct, sound cement grout, coatings and other internal barriers in the prestressing steel and good anchorage protection measures (for example, encapsulated systems). However, potential weak links exist, among others, when the high strength concrete has high permeability (due to mix design and construction or due to service/exposure conditions), and when the concrete has cracking (due to shrinkage or service loading). Additional weak links occur when ducts are not adequately spliced or adequately protected by impermeable concrete and so are prone to severe corrosion. In addition, voids, bleed water and cracks can be present in the Portland cement grout. Finally, the prestressing steel may not be adequately protected and handled during construction, including inadequate or total lack of temporary protection techniques.

One of the major problems that agencies face today is the difficulty of providing good monitoring and inspection techniques for bonded post-tensioned structures. Condition surveys are often limited to visual inspections for signs of cracking, spalling and rust staining. This limited technique can often overlook the deterioration of prestressing steel and fail to detect the potential for very severe and sudden collapses, as demonstrated in the Ynys-y-Gwas Bridge failure.[2] Therefore, as stated by West et al.[30] "…there is fear that figures reporting the incidence of corrosion in prestressed structures based on limited or visual inspections may be unconservative and produce a false sense of security." The same could happen even when using advanced techniques as borescopy, since in this case the analysis is limited to specific areas in selected bridge elements. Grout voids or even corrosion of prestressing steel in many areas of the bridge element may still be overlooked.

As stated by Ganz[31] the design of the corrosion protection systems should take into account that most parts of the tendons are not accessible during the design life, and that in general individual components or the entire tendons, are not replaceable. Even if special details are provided to allow replaceability of the tendons during the design life, the replacement should be carried out only in "emergency" situations.

Stable grout mixes and better grouting procedures are now being implemented, in part after the important findings of Schockker et al.[32] at the Phil M. Ferguson Structural Engineering Laboratory (FSEL) at the

University of Texas at Austin, in 1999. The American Segmental Bridge Institute (ASBI) in 2001 launched the Grouting Certification Training Program, which was first held in August 2001, at FSEL. This training program has been adopted by various Transportation Agencies throughout the United States as a requirement for grouting supervisors and inspectors.[1,18] Additionally, the Post-Tensioning Institute Committee on Grouting Specification, published in February 2001 the "Specification for Grouting of Post-Tensioned Structures."[19] The use of this guide and the new and better inspection procedures are expected to yield even more durable structures.

Besides high performance grouts and better grouting practices, plastic ducts either polyethylene (PE) or polypropylene (PP) are being implemented as reported by *fib* technical report on "Corrugated Plastic Ducts for Internal Bonded-Post-Tensioning."[25] Yet, many durability aspects of bonded internal post-tensioning systems require further research and analysis.

1.2 DESIGNING BONDED POST-TENSIONED SYSTEMS FOR DURABILITY

Designing for durability requires the same thought process as design for any other limit state or form of structural loading. The engineer must first assess the type of loading to be considered and determine its intensity. Then, the engineer must determine the effects of the loading on the structure and design the structure to resist the loading through careful proportioning and detailing. The various components of the structure may have different design requirements depending on their function, and these requirements must be identified and addressed. A simplified analogy between durability design and design for structural loading is illustrated in Figure 1.1. Although the two processes are similar, the "precision" of design for durability can be significantly different from design for structural loading. The types and intensities of design requirements or loading can be assessed with similar accuracy in both cases. However, the resistance of the structure to durability attack can not be determined with the same level of certainty as in the estimation of the resistance of the structure to structural loading. This lack of precision is reflected in the durability design process:

Durability design guidelines should provide the engineer with the following information:

- How to determine when different forms of attack on durability should be considered.

  The engineer should be able to establish when durability must be considered as a limit state in the design process and be able to identify which forms of attack will occur in a given situation. Due to the varied climate, geology and geography of any project location, durability may play a significant role in the design process for some situations, while in others it may not.

- How to evaluate the severity of attack on structural durability in a given situation.

  Once it has been determined that certain forms of durability problems may occur, the possible severity of attack needs to be assessed.

- How to determine what level of protection is necessary for the various components of the structure.

  The required level of protection for the structural components is a function of the forms and severity of attack that may be encountered in a particular situation. It is also strongly affected by the susceptibility of the various components of the substructure to the expected forms of attack.

- What measures can be employed to provide the necessary level of protection.

  Once the required level of protection has been determined, the engineer should be presented with design options to provide the necessary level of protection for durability.

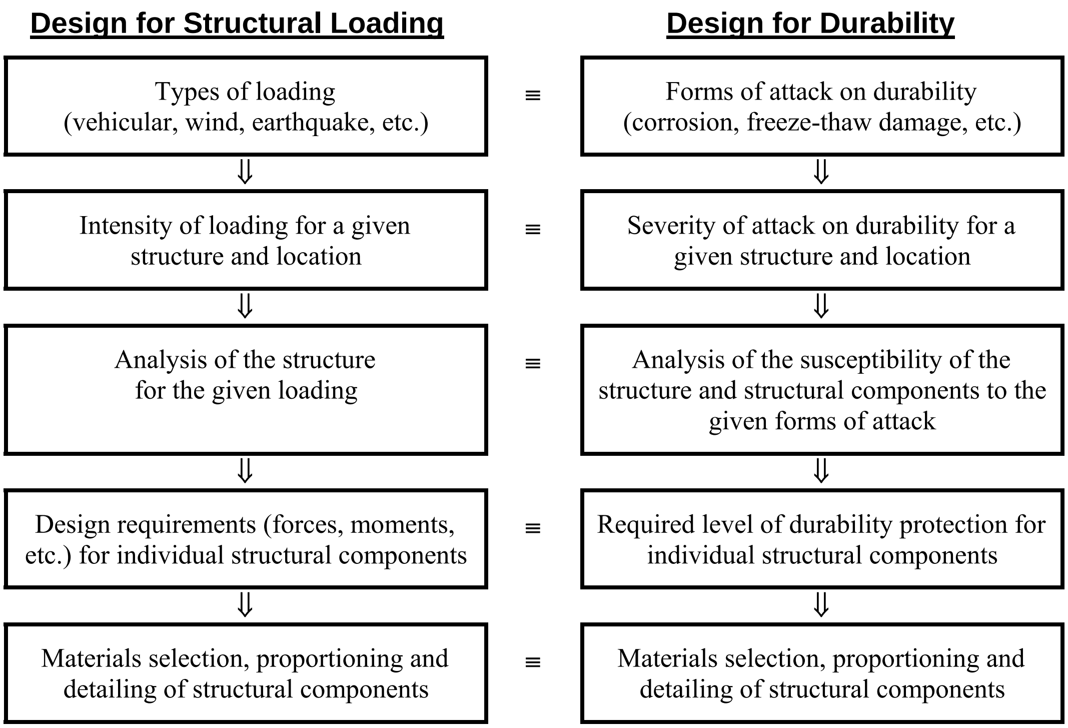


Figure 1.1 Simplified Analogy between Design for Structural Loading and Durability

1.3 RESEARCH OBJECTIVES AND SCOPE

1.3.1 Project Statement

This report contains the Final Conclusions, Recommendations and Design Guidelines for Corrosion Protection, as part of the University of Texas at Austin, Center for Transportation Research Project 0-1405: "Durability Design of Post-Tensioned Bridge Substructure Elements." The project and its extension into project TxDOT 0-4562 is performed at the Phil M. Ferguson Structural Engineering Laboratory and it is sponsored by the Texas Department of Transportation and Federal Highway Administration. The title of Project 0-1405 involves two main aspects:

- Durability of Bridge Substructures, and
- Post-Tensioned Bridge Substructures.

The substructure emphasis is in response to the deteriorating condition of bridge substructures in some areas of Texas. While considerable research and design effort has been given to bridge deck design to prevent corrosion damage, substructures had historically been more overlooked. Often superstructure drainage details result in substructures having a high exposure to aggressive agents such as, deicing salts, also substructures are often in direct contact with salt water and damaging soils.

The second aspect of the research is post-tensioned substructures. Relatively few post-tensioned substructures have been used in the past. There are many possible applications in bridge substructures where post-tensioning can provide structural and economical benefits, and can possibly improve durability. Post-tensioning is now being used in Texas bridge substructures, and it is reasonable to expect the use of post-tensioning to increase in the future as precasting of substructure components becomes more prevalent and as foundation sizes increase. This is expected, even though some problems have been encountered in post-tensioned bridges throughout the world as mentioned previously. The same lessons learned are generally applicable to post-tensioned bridge superstructures. Thus all results can be considered as applicable to post-tensioned bridges in general.

The problem that bridge engineers face is that there are few comprehensive durability design guidelines for post-tensioned concrete structures. Durability design guidelines should provide information on how to identify possible durability problems, how to improve durability using post-tensioning, and how to ensure that the post-tensioning system does not introduce new durability problems.

1.3.2 Project Objectives

The overall research objectives for TXDOT Project 0-1405 are as follows:

1. To examine the use of post-tensioning in bridge substructures,
2. To identify durability concerns for bridge substructures in Texas,
3. To identify existing technology to ensure durability or improve durability,
4. To develop experimental testing programs to evaluate protection measures for improving the durability of post-tensioned bridge substructures, and
5. To develop durability design guidelines and recommendations for post-tensioned bridge substructures. (generalized to post-tensioned bridges)

A review of literature has indicated that while a few problems have been encountered in some bridges in Europe, Japan, and the U.S.A., damage has been limited to a very small percentage of post-tensioned bridges. In general, post-tensioning systems have been successfully used in bridge designs. However, as these bridges age and increase in cumulative exposure, more problems are being noted. New practices and materials are required to guarantee the safety and design life of these structures.

The initial literature review performed by West[33] identified a substantial amount of relevant information that could be applied to the durability of post-tensioned bridge substructures. This existing information allowed the scope of the experimental portion of the project to be narrowed. The final objective represents the culmination of the project. All of the research findings are to be compiled into the practical format of comprehensive durability design guidelines.

1.3.3 Project Scope

The subject of durability is extremely broad, and as a result a broad scope of research was developed for TXDOT Project 0-1405. Based on the project proposal and an initial review of relevant literature performed by West[33], the project scope and necessary work plan were defined. The main components of TXDOT Project 0-1405 are:

1. Extensive Literature Review
2. Survey of Existing Bridge Substructures Inspection Reports (BRINSAP)
3. Long-Term Corrosion Tests with Large-Scale Post-Tensioned Beam and Column Elements
4. Investigation of Corrosion Protection (near joints) for Internal Prestressing Tendons in Precast Segmental Bridges
5. Development of Improved Grouts for Post-Tensioning
6. Development of recommendations and design guidelines for durable bonded post-tensioned bridges

Components 1 and 2 (literature review and survey of Brinsap report) were performed initially by West,[33] Schokker,[34] Koester[35] and Larosche[36] and findings up to 1998 were published in References 33 and 34. The literature review process was continued by Salas[37] and is reported in Reference 37.

Component 3 was divided into Large Scale Column Corrosion Tests and Large Scale Beam Corrosion Tests. The column tests were started by Larosche[36] and West.[33] Column exposure testing began in July 1996. The beam tests were implemented in two phases: the first phase was implemented by West,[33] and exposure testing began in December 1997. The second phase was implemented by Schokker,[34] and exposure testing begun in December 1998. Comprehensive autopsies of around half of these specimens, at the end of their exposure testing period were performed by Kotys[38] and Salas.[37] The remaining specimens will undergo several years until comprehensive autopsies are performed, which will be done under TxDOT Project 0-4562.

Component 4 (corrosion protection at joints of segmental bridges) was developed and implemented by Vignos[39] under TxDOT Project 0-1264. This testing program was transferred to TxDOT Project 0-1405 in 1995 for long-term testing. Although this aspect of the research was developed under Project 0-1264 to address corrosion concerns for precast segmental bridge superstructures, the concepts and variables are equally applicable to precast segmental substructures, and the testing program fits well within the scope of Project 0-1405. Half of the macrocell laboratory specimens were autopsied at four and a half years of exposure testing by West.[33] Final autopsies of the remaining specimens were performed by Salas[37] and Kotys.[37]

Component 5 (Development of Improved Grouts for Post-Tensioning) was developed and implemented by Schokker[34] based on previous work published by Hamilton[40] and Koester.[35] The accelerated corrosion testing was performed and conclusions were drawn and published.[32, 34] Under this portion of the research, high-performance grouts for bonded post-tensioning were developed through a series of fresh property tests, accelerated corrosion tests, and large-scale field trials. These grouts have become widely used in practice.

Component 6 (development of recommendations and design guidelines for durable bonded post-tensioned bridge substructures) refers to the most important implementation directed aspect of the research program. Interim design guidelines including various forms of durability attack were developed and published by West and Schokker[41] based on research results up to 1999. Updated Guidelines concentrating on corrosion risk and based on final autopsy results from the macrocell, column and beam tests are included in this report.

The project scope is outlined in Figure 1.2. This figure shows the cooperative effort performed by all graduate research assistants during the length of the project. In this figure the years in brackets show the actual publication dates for each Technical Report, published under TxDOT Project 0-1405. A more detailed description of the involvement of each graduate research assistant is shown in Table 1.1.

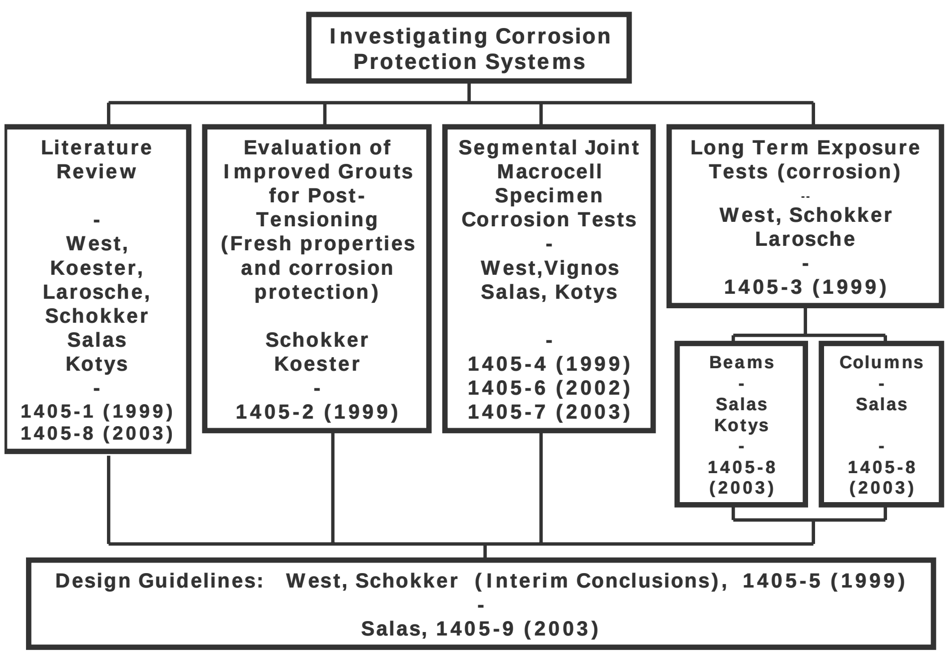


Figure 1.2 TxDOT Project 0-1405 Scope, Researchers and Technical Reports

Table 1.1 Major Project Tasks and Contributions of Graduate Students

Project Task	West (Ref. 33)	Schokker (Ref. 34)	Koester (Ref. 35)	Larosche (Ref. 36)	Vignos (Ref. 39)	Kotys (Ref. 38)	Salas (Ref. 37)
Literature Review	X	X	X	X			X
Identification of Substructure PT Applications	X		X				
Identification of Protection Variables	X		X				
Survey of Existing Structures				X			
Testing Program Design							
Long Term Beam Exposure Tests	X						
Long Term Column Exposure Tests	X			X			
Segmental Macrocell Corrosion Tests					X		
Evaluation of Improved Grouts for PT		X	X				
Fabrication of Test Specimens							
Long Term Beam Exposure Tests	X	X					
Long Term Column Exposure Tests	X			X			
Segmental Macrocell Corrosion Tests					X		
Evaluation of Improved Grouts for PT		X	X				
Initial Exposure Testing							
Long Term Beam Exposure Tests	X	X					
Long Term Column Exposure Tests	X	X					
Segmental Macrocell Corrosion Tests	X				X		
Evaluation of Improved Grouts for PT		X	X				
Limited Specimen Autopsies							
Long Term Beam Exposure Tests		X					
Long Term Column Exposure Tests		X					
Segmental Macrocell Corrosion Tests	X						
Continuation of Exposure Testing							
Long Term Beam Exposure Tests						X	X
Long Term Column Exposure Tests						X	X
Segmental Macrocell Corrosion Tests							X
Final Autopsies							
Long Term Beam Exposure Tests						X	X
Long Term Column Corrosion Tests							X
Segmental Macrocell Corrosion Tests						X	X
Continuation of Exposure Testing							
Long Term Beam Exposure Tests						X	X
Preliminary Design Guidelines	X	X					
Design Guidelines update							X

1.3.4 Project Reports

Nine reports were scheduled to be developed from Project 0-1405 as listed in Table 1.2. This report is the ninth in this series.

Report 0-1405-1 provides a detailed background on the topic of durability design of post-tensioned bridge substructures. The report contains an extensive literature review on various aspects of the durability of post-tensioned bridge substructures and a detailed analysis of bridge substructure condition rating data in the State of Texas.

Report 0-1405-2 presents a detailed study of improved and high-performance grouts for bonded post-tensioned structures. Three testing phases were employed in the testing program: fresh property tests, accelerated corrosion tests and large-scale pumping tests. The testing process followed a progression of the three phases. A large number of variables were first investigated for fresh properties. Suitable mixtures then proceeded to accelerated corrosion tests. Finally, the most promising mixtures from the first two phases were tested in the large-scale pumping tests. The variables investigated included water-cement ratio, superplasticizer, antibleed admixture, expanding admixture, corrosion inhibitor, silica fume and fly ash. Two optimized grouts were recommended depending on the particular post-tensioning application.

Report 0-1405-3 describes the development of two long-term, large-scale exposure testing programs, one with beam elements, and one with columns. A detailed discussion of the design of the test specimens and selection of variables is presented. Preliminary experimental data is presented and analyzed, including cracking behavior, chloride penetration, half-cell potential measurements and corrosion rate measurements. Preliminary conclusions are presented.

Report 0-1405-4 describes a series of macrocell corrosion specimens developed to examine corrosion protection for internal prestressing tendons in precast segmental bridges. This report briefly describes the test specimens and variables, and presents and discusses four and a half years of exposure test data. One-half (nineteen of thirty-eight) of the macrocell specimens were subjected to a forensic examination after four and a half years of testing. A detailed description of the autopsy process and findings is included. Conclusions based on the exposure testing and forensic examination are presented.

Report 0-1405-5 contains a summary of the conclusions and recommendations from the first four reports from Project 0-1405. The findings of the literature review and experimental work were used to develop preliminary durability design guidelines for post-tensioned bridge substructures. The durability design process is described, and guidance is provided for assessing the durability risk and for ensuring protection against freeze-thaw damage, sulfate attack and corrosion of steel reinforcement.

Report 0-1405-6 describes a series of macrocell corrosion specimens developed to examine corrosion protection for internal prestressing tendons in precast segmental bridges. This report briefly describes the test specimens and variables, and presents and discusses eight years of exposure test data. One-half (nineteen of thirty-eight) of the macrocell specimens were subjected to a forensic examination after four and a half years of testing, and were reported in Report 1405-4. A detailed description of the autopsy process for the remaining macrocell specimens and findings is included. Final conclusions and recommendations based on the exposure testing and forensic examination are presented.

Report 0-1405-7 describes a series of beam corrosion specimens developed to examine corrosion protection for bonded internal prestressing tendons in linear flexural bridge elements. This report briefly describes the test specimens and variables, and presents and discusses the results after approximately one-half of the beam specimens were autopsied after three an a half years and four a half years of exposure testing. A detailed description of the autopsy process and findings is included. Final conclusions based on the exposure testing and forensic examination are presented. The report concludes with recommendations for materials and implementation measures.

Report 0-1405-8 describes a series of column corrosion specimens developed to examine the effect of post-tensioning on concrete pier and column durability (corrosion protection) through precompression of the concrete and precompression of construction joints, and to investigate the relative performance of various aspects of corrosion protection for post-tensioning, including concrete type, duct type, post-tensioning bar coatings and loading. A detailed description of the autopsy process and findings is included. Final conclusions based on the exposure testing and forensic examination are presented. The report concludes with recommendations for materials and implementation measures.

Report 0-1405-9 contains a summary of the final conclusions and recommendations from Project 0-1405, concentrating on corrosion protection. The findings of the literature review and experimental work were used to develop corrosion protection design guidelines for post-tensioned bridge substructures. The durability design process is described, and guidance is provided for assessing the corrosion risk of steel reinforcement. Conclusions related to other durability aspects are included in Report 1405-5.

Table 1.2 Proposed Project 0-1405 Reports

Number	Title	Estimated Completion
0-1405-1	State of the Art Durability of Post-Tensioned Bridge Substructures	1999
0-1405-2	Development of High-Performance Grouts for Bonded Post-Tensioned Structures	1999
0-1405-3	Long-term Post-Tensioned Beam and Column Exposure Test Specimens: Experimental Program	1999
0-1405-4	Corrosion Protection for Bonded Internal Tendons in Precast Segmental Construction	1999
0-1405-5	Interim Conclusions, Recommendations and Design Guidelines for Durability of Post-Tensioned Bridge Substructures	1999
0-1405-6	Final Evaluation of Corrosion Protection for Bonded Internal Tendons in Precast Segmental Construction	2002
0-1405-7	Long-term Post-Tensioned Beam Exposure Test Specimens: Final Evaluation	2003
0-1405-8	Long-term Post-Tensioned Column Exposure Test Specimens: Final Evaluation	2003
0-1405-9	Conclusions, Recommendations and Design Guidelines for Corrosion Protection of Post-Tensioned Bridges	2003
0-1405-S	Corrosion Protection of Post-Tensioned Bridge Elements	2003

Several dissertations and theses at The University of Texas at Austin were developed from the research from Project 0-1405. These documents may be valuable supplements to specific areas in the research and are listed in Table 1.3 for reference.

Table 1.3 Project 0-1405 Theses and Dissertations, The University of Texas at Austin

Title	Author	Date
Masters Theses		
Evaluation of Cement Grouts for Strand Protection Using Accelerated Corrosion Tests"	Bradley D. Koester	12/95
"Durability Examination of Bonded Tendons in Concrete Beams under Aggressive Corrosive Environment"	Andrea L. Kotys	5/03
"Test Method for Evaluating Corrosion Mechanisms in Standard Bridge Columns"	Carl J. Larosche	8/99
"Test Method for Evaluating the Corrosion Protection of Internal Tendons Across Segmental Bridge Joints"	Rene P. Vignos	5/94
Ph.D. Dissertations		
"Accelerated Corrosion Testing, Evaluation and Durability Design of Bonded Post-Tensioned Concrete Tendons"	Ruben M. Salas	8/03
"Improving Corrosion Resistance of Post-Tensioned Substructures Emphasizing High-Performance Grouts"	Andrea J. Schokker	5/99
"Durability Design of Post-Tensioned Bridge Substructures"	Jeffrey S. West	5/99

CHAPTER 2: INTERNAL BONDED POST-TENSIONING SYSTEMS

2.1 PRESTRESSED CONCRETE

2.1.1 Historical Development

Key events in the history of prestressed concrete structures started at the end of the 19th Century, with the concept of imposing preservice stresses on hardened concrete. However, the most important event in the development of this technique was recorded in 1933, when E. Freyssinet demonstrated the advantage of using higher strength concrete and high strength steel to minimize losses. Table 2.1 summarizes the key events as reported by Schupack.[42]

Table 2.1 Key Developments in Prestressed Concrete History[42]

Year	Author/Researcher	Description
1888	P.H.Jackson	Concept of imposing preservice stresses on hardened concrete
1907	M. Koenen	Identify losses due to classic shortening
1908	G.R. Steiner	Recognized losses due to shrinkage
1928	F.Dischinger	Loss of prestress compensated by retensioning
1933	E.Freyssinet	Demonstrated advantage of using higher strength concrete and high strength steel to minimize losses
1939	K.Wettstein E.Hoyer	Used high strength piano wire
1943	J.M.Crom	Used high tensile drawn wire for tanks and pipe
1944	G.Magnel	Identified the relaxation losses of work-hardened steels under constant strain
1950	Reported by W.O. Everling.	Use of stress relieved wire 240 ksi (1.65 GPa) and strand 250 ksi (1.72 GPa) to provide user friendly steel
1963	T.Cahill	Developed low-relaxation steel reducing loss from about 12 to 2.5%

After these events, no other dramatic new concepts were reported through the end of the 20th Century, but continuous development has occurred and prestressing steel usage has been significantly increasing since the 1960s.

After the second world war, as reported by Godart,[3] countries like France experienced the construction euphoria of widespread infrastructure reconstruction, during a period marked by cement and steel shortage. French viaducts were reconstructed with simply supported spans made up of prestressed beams, known as VIPP (Viaducts a travées Indépendantes a Poutres Précontraintes: Viaducts with simply supported spans made of prestressed beams). After an investigation by Trouillet in the year 2000 as reported by Godart,[3] fifteen out of a total of 720 VIPP built before 1966, had been demolished because of tendon corrosion.

In the USA, according to the National Bridge Inventory Database,[43] there are approximately 600,000 bridges, of which half were built between 1950 and 1994. Based on Federal Highway Administration (FHWA) data reported by Yunovich,[43] approximately 18.5% of the total are prestressed concrete bridges. These include both post-tensioned and pretensioned technology.

The beginning of post-tensioned concrete bridges in the US started in Madison County, Tennessee,[44] where a concrete highway bridge was built and opened to traffic on October 28, 1950. The bridge consisted of three-spans with a total length of 81 feet, using precast concrete blocks with mortar joints. This bridge was shortly followed by the Walnut Lane Bridge in Philadelphia, completed in the fall of 1950 and opened to traffic in February 1951. The construction of the Walnut Lane Bridge started in 1949, and therefore the beginning of post-tensioning in the US is often associated with this bridge. Three segmental I-girder bridges were constructed in western New York in 1950 and 1951.[6] By the end of 1952, there were prestressed concrete bridges in eight states and this grew to 17 states in 1954. By the year 1985, there were more than 60,000 prestressed concrete bridges in the United States.[44] The majority of these bridges used pretensioned units. The first precast post-tensioned segmental box girder bridge was the John F. Kennedy Memorial Causeway, built in 1971 on the coast of Texas. The Pine Valley Creek Bridge, built in the coastal mountains near San Diego, California in 1974 was the first U.S. cast-in place balanced cantilever bridge.[6]

In 1999, the use of bonded post-tensioning steel in bridges and earthwork comprised 22 percent (about 29,000 tons) of the total post-tensioning steel tonnage used in the US. Other uses include buildings, slabs-on-ground and miscellaneous.[6]

2.1.2 Basic Definitions

The current research refers to bonded internal post-tensioning systems. In general, prestressed concrete can be pretensioned or post-tensioned, depending on when the prestressing steel is tensioned. Table 2.2 gives the basic definitions and main differences among the different prestressing techniques.

In bridge post-tensioning applications, prestressing steels often consist of a bundle (tendon) of 7-wire prestressing strands of very high strength steel that are run through the ducts previously placed into the concrete element before casting. The end anchorage includes wedges to hold the tendons that run through an anchor head with tapered holes. Figure 2.1 shows a typical anchorage system. The wedges contain teeth that bite into the strand, and this makes corrosion protection of the end anchorage area critical.[34] Once the prestressing steel has been stressed to the desired level and adequately anchored, the ducts can be injected with cement grout, in which case the system is referred as bonded. In this case, the grout develops bond between the steel tendon and the duct and surrounding concrete and acts as a corrosion barrier.

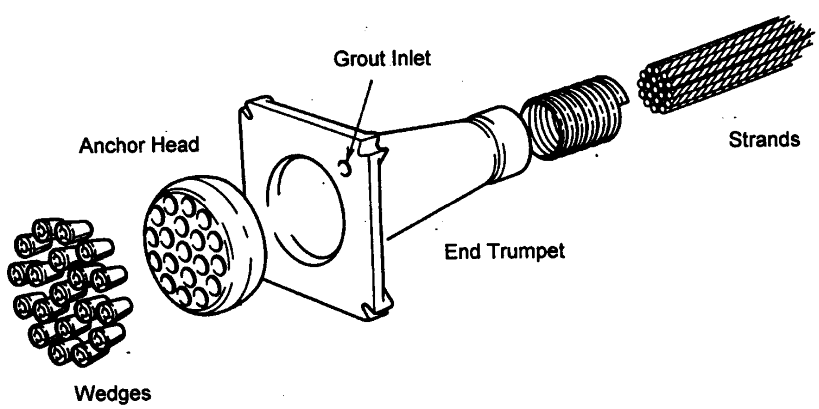


Figure 2.1 Post-Tensioning Anchorage Details (from T.Y. Lin, and N.H. Burns, 1981)[34]

Table 2.2 Basic Definitions (Adapted from Reference 44)

Term	Definition
Prestressed Concrete	Technique of tensioning the steel reinforcement in a reinforced concrete structures so as to place the concrete in a state of compression to counteract the tensile stresses resulting from service loads. Prestressed concrete may be pretensioned or post-tensioned, depending on when the prestressing steel is tensioned.
Pretensioned Concrete	Pretensioned concrete is made by stressing the steel (normally wires or strand) between fixed points, usually the ends of a rigid casting bed, and then casting concrete around the steel. After the concrete has gained sufficient strength, the strands are released from their original anchorage, thereby applying compressive stress to the concrete.
Post-Tensioned Concrete	In post-tensioned concrete, the tendons are tensioned after the concrete has gained strength. This can be done by using tendons that have been encased in sheathing during manufacture. These covered tendons are cast in the concrete. Alternatively, ducts are cast in the concrete, normally using metal sheathing, through which the tendons are later threaded. When the concrete has achieved a predetermined strength, the tendons are stressed and anchored. Post-tensioned construction can be classified as bonded or unbonded.
Unbonded construction.	In unbonded construction, the tendons transfer stress to the structure only at the anchorages. Except for external tendons, unbonded construction is very rarely used in highway bridges because of the severity of the service environment and the uncontrolled crack width and reduction in capacity at the ultimate limit state.
Bonded construction	In bonded construction, grout is injected to fill the void between the tendon and the duct. This not only protects the tendon against corrosion but also increases the ultimate strength capacity of the component.
Internal Tendons	Tendons that are embedded in a member.
External Tendons	Tendons most frequently used in cellular sections and must be unbonded or partially bonded at intermediate deviators.

2.1.3 Advantages and Disadvantages of Internal and External Post-Tensioning

Prestressed concrete benefits include improved crack control (higher cracking moment, fewer cracks, smaller crack widths), reduced reinforcement congestion, continuity of reinforcement and efficient use of high strength steel and concrete. Additionally, post-tensioning will allow for a quick and efficient joining of precast elements and continuity between existing components and additions.[34]

Post-tensioning systems can be internal and external. Table 2.3 shows a comparison of these systems in terms of their advantages and disadvantages.

Table 2.3 Advantages and Disadvantages of Internal and External Post-Tensioning (Adapted from References 3 and 46)

	Advantages	Disadvantages
Internal Post-Tensioning	- allow for possible reanchoring of the strands, losing only locally the prestress force in the event of failure of a section - tendons can follow well the moment diagram - low production costs - system reserves available	- concentration of vent hoses at the road surface (can be avoided) - design experience necessary - high requirements to quality, especially during grouting - grout characteristics are critical - expensive maintenance - very difficult to investigate and impossible to replace
External Post-Tensioning	- webs free of tendons - low weight - high quality - no vent hoses - tendons with high level-corrosion protection - restressable - strengthenable - theoretically replaceable - more easy to investigate	- can be damaged - exposed to atmospheric influences - reserve is missing due to no bond - full loading action on the anchorage - critical assembly operations - more expensive construction and demolition - more sensitive to fire - more easily attackable by vandals - with any failure the prestressing force disappears over the overall length of the tendon - the buckling and whipping generated by a sudden rupture create risks for the inspection staff and for the other tendons - design experience necessary

2.1.4 Mixed Post-Tensioning

There has been world wide debate with respect to the use of either internal or external post-tensioning. After analyzing the advantages and disadvantages for each system, it appears obvious that in many cases the solution would be in the intermediate area. As noted by Jungwirth and Gehlen[46] a "mixed construction," which is use of a combination of internal and external tendons in a section, is considered to have many advantages. These include among others: different loading cases during the construction and demolition phase can be handled more easily with internal post-tensioning tendons; residual load bearing capacity in case of failure of external tendons (for example: in case of fire or earthquakes); possibility of a subsequent strengthening with external tendons; and , better ductility with internal tendons in case of earthquakes.

After considering some decisions in Germany, Virlogeux [47] stated in 1999 that "… stopping internal prestressing, or limiting its application by some recommendations issued to protect against some exceptional problems, could create difficulties everywhere, especially when coming from Germany, which has the very well-deserved reputation of its use of prestressing. It would be against the facts: hundreds of thousands of structures have been built in prestressed concrete on the five continents, with very few problems."

Chaussin[48] stated in Ghent that "…it is a good thing, whenever it is possible, not to put all one's eggs in one basket and to combine internal and external tendons." He recommends that in internally prestressed members, a particular attention must be given to anchorage areas. The risk could be counteracted by placing in these zones enough non-prestressed mild reinforcing steel to withstand the action effects susceptible to occur even if cracks appear. In that case, crack formation would constitute an evident sign of distress for an experienced eye and the structure manager would have enough time to take appropriate actions. In externally prestressed structures, structures would have to be designed so the failure of one tendon does not seriously impair the proper functioning of the structure. The broken tendon should be easily inspected and replaced. In this type of structure, it is advisable to incorporate, during the design phase, the potential for additional external tendons (anchor and deviation points) for strengthening purposes, either to respond to changes in service conditions or to compensate the loss of part of the existing prestressing or to counteract an unexpected behavior of the structural concrete. It is understood that the additional cost of such measures if they are included in the design phase, would be a small fraction of the cost of installing them later during the structure's service life.

2.1.5 Applications of Bonded Post-Tensioning

Bonded post-tensioning is currently used in many bridge and building applications. In bridge applications, bonded post-tensioning is used in:

- Precast segmental balanced cantilever construction (cantilever tendons and continuity tendons) See Figure 2.2.

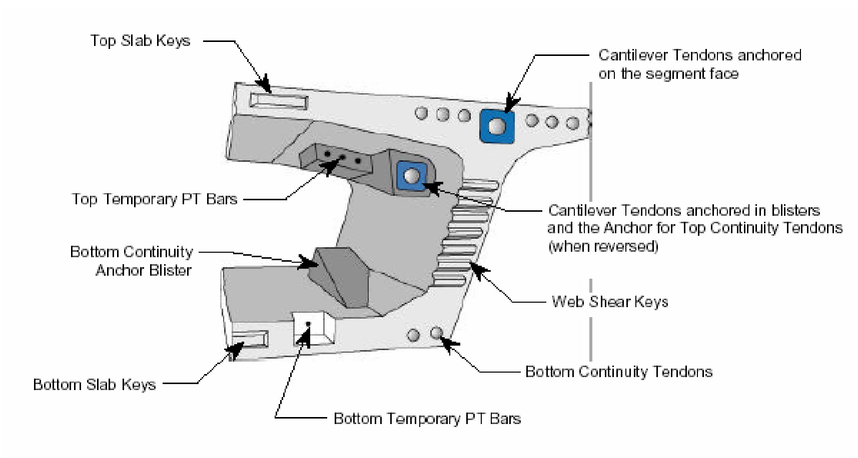


Figure 2.2 Typical Balanced Cantilever Segment[16]

- Precast segmental span-by-span construction
- Post-tensioned AASHTO, bulb-T, and spliced girders
- Cast-in Place segmental balanced cantilever construction
- Cast-in-place bridges on falsework
- Transverse post-tensioning of superstructures (transverse top slab post-tensioning, transverse post-tensioning in diaphragms, vertical post-tensioning in diaphragms, transverse post-tensioning in deviator ribs of precast segments, vertical post-tensioning bars in webs,
- Post-tensioning of substructures, see Figure 2.3 and Figure 2.4 (hammerhead piers, straddle bents, cantilever piers, precast box piers, precast I-section pier columns, transverse confinement tendons at tops of piers).

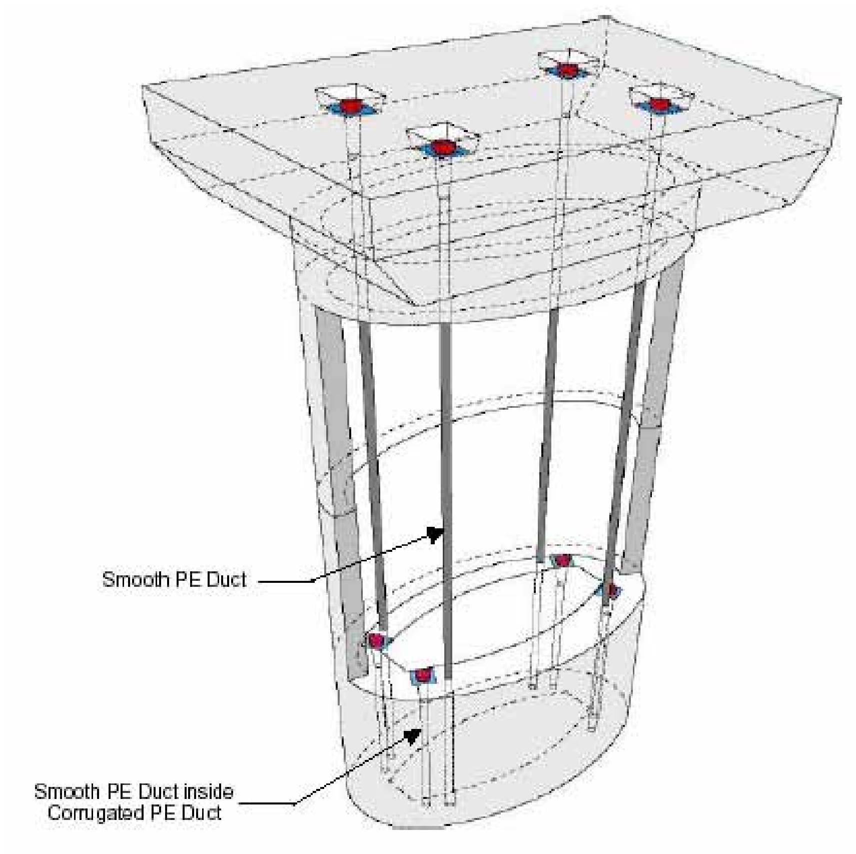
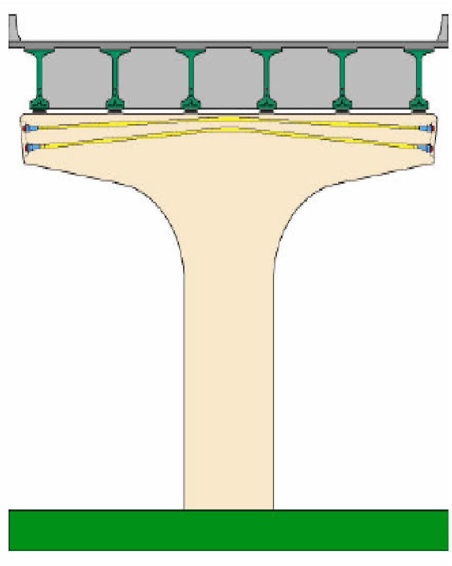
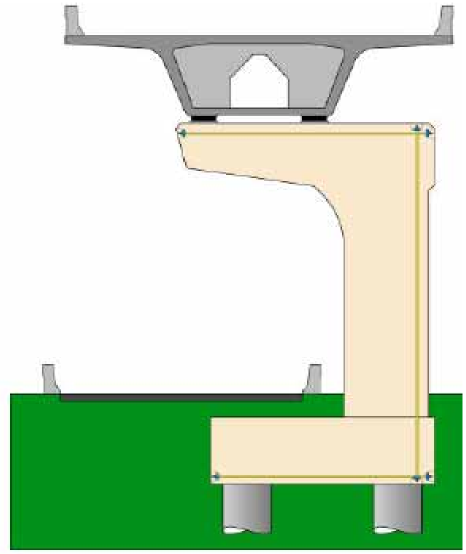


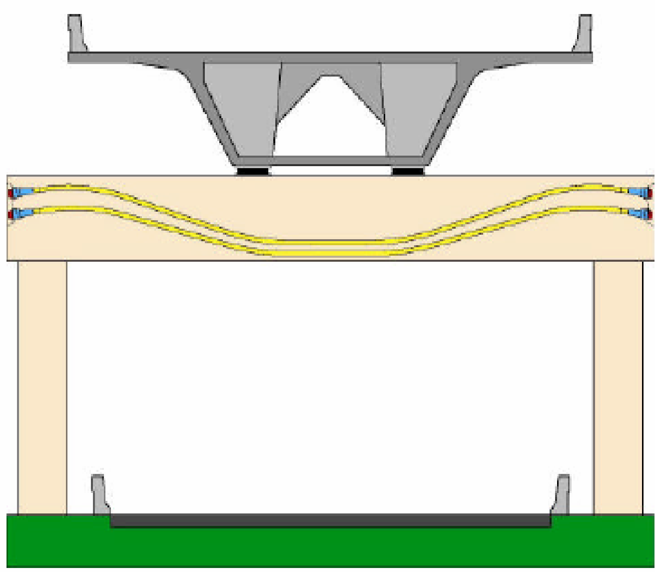
Figure 2.3 Vertical Post-Tensioning of the High-Level Approach Piers of the Sunshine Skyway Bridge, Florida[16]



(a)



(b)



(c)

Figure 2.4 Post-Tensioning in (a) Hammerhead Piers, (b) Cantilever Piers, and (c) Straddle Bents[16]

A detailed description of the above applications is included in Volume 1, Chapter 2, of the document "New Directions for Florida Post-Tensioned Bridges," published in 2002 by the Florida Department of Transportation.[16]

2.1.6 Mixed Reinforcement

AASHTO LRFD (Load and Resistance Factor Design) Bridge specification explicitly recognized the use of partial prestressing,[49] an unfortunate term. There are several conflicting definitions for partial prestressing in use in practice. It may refer to the level of stresses and what degree of tensile stress is permitted in the precompressed tensile zone under service load. AASHTO is using the term partial prestressed structures to refer to structural concrete members with a combination of high strength prestressing steel and non-prestressed mild steel reinforcement. Internationally the preferred term for this use is 'mixed reinforcement' a combination of active reinforcement (stressed in construction) and passive reinforcement (stressed by subsequent load condition). This more correct terminology will be used in this study. The relative amounts of prestressing steel and reinforcing bars may vary, and the level of prestress in the prestressing steel may be altered to suit specific design requirements. In most cases, members with mixed reinforcement are expected to crack under service load conditions (flexural cracks due to applied loads).[33]

As stated by West,[33] in the past practice in the United States prestressed concrete elements have always been required to meet the classic definition of full prestressing at service load levels where concrete stresses are kept within allowable limits and members are generally assumed to be uncracked (no flexural cracks due to applied loading). The design requirements for prestressed concrete were distinctly separate from those for reinforced concrete (non-prestressed) members, and are located in different chapters or sections of the codes. The fully prestressed condition may not always lead to an optimum design. The limitation of concrete tensile stresses to below cracking can lead to large prestress requirements, resulting in very conservative design, excessive creep deflections (camber) and the requirement for staged prestressing as construction progresses. Some relief was given by allowing some levels of tensile stress in the precompressed tensile zones at service load levels.

Mixed reinforcement can provide a desirable design alternative to reinforced concrete and fully prestressed designs in many types of structures, including bridge substructures. The opposition to mixed reinforcement designs and the reluctance to recognize higher allowable tensile stresses at service level in design codes has primarily been related to concerns for increased cracking and its effect on corrosion. Mixed reinforcement structures will generally have more cracks than comparable fully prestressed structures but lesser cracking than non-prestressed structures. No explicit crack width limitations are placed on non-prestressed members. Due to the widely accepted notion that prestressing steel is more susceptible to corrosion, and that the consequences of corrosion in prestressed elements are more severe than in reinforced concrete, many engineers have felt that the benefits of mixed reinforced are outweighed by the increased corrosion risk. Little or no research has been performed to assess the effect of mixed reinforcement on corrosion.[33]

2.2 DURABILITY OF BONDED POST-TENSIONED CONCRETE STRUCTURES

As referenced by Kuesel,[50] Hardy-Cross said that "The first requirement for a beautiful bridge is that it must stand up long enough for us to look at it." Kuesel continue saying that "…We should endeavor to design bridges that will be functional, enduring and – yes – beautiful. …. Design codes should require consideration of endurability – inspectability, maintainability, reliability and resistance to water, corrosion, temperature cycles and neglect. …we should not be debating how much further we can reduce safety margins (for loads and strength), but rather how much we should increase safety margins (for wear and corrosion). … All around us we see the results of neglect of long-term problems, and yet we persist in concentrating our attention on short-term design. Least first cost is a short –term objective. Long useful life is a long term objective. We need them both."

Godart[3] indicates that "…Design defects are … mainly linked to construction defects or unsuitable techniques or to the use of low durability materials. The two principal construction defects responsible for corrosion are poor waterproofing and incomplete grouting of the prestressing ducts. In addition to these defects there is poor sealing of end anchorages, deck anchorages and transverse anchorages."

2.2.1 *Problems Encountered Around the World: The Ghent Workshop*

In November 2001, engineers from many countries gathered at Ghent University, under the sponsorship of *fib* and IABSE, to discuss the "Durability of Post-Tensioning Tendons." The findings are contain in the *fib*-IABSE Technical Report, Bulletin 15.[11] The problems reported from different countries lead to similar findings. Zivanovic et al.[51] succinctly summarized the conference findings with respect to the inventory and condition of post-tensioning bridges as follows:

a) Design Defects:

- Lack of waterproofing;
- Lack of sealing behind tendon anchorages;
- Construction resulting in a large number of unprepared construction joints, which could give rise to cracking due to restrained shrinkage;
- Use of unprotected transverse tendons in grooves in the deck;
- Use of sheaths made from bitumen-coated Kraft paper wrapped around the tendons which made grouting impossible;
- Low ratio of rebar/total steel, as low as zero for longitudinal construction joints along the length of the deck between the beams and slabs;
- Large numbers of tendons per span in older structures with deck anchorages increasing the number of points of possible water ingress;
- Use between 1950 and 1970 of prestressing steel which was susceptible to stress corrosion which gave rise to the possibility of brittle fracture;
- Lack of provision for drainage
- Leaking expansion joints;
- Insufficient concrete cover over the reinforcement, resulting in corrosion of the reinforcement and spalling of the concrete giving easier access to the prestressing tendons for aggressive agents.

b) Construction defects

The main defects leading to corrosion are poor waterproofing, incomplete grouting of the prestressing ducts and poor sealing of end anchorages.

Although concrete has generally performed well in older structures, there may be areas where poor workmanship has given rise to honeycombing or shrinkage cracking. The most common location are the soffits of flanges where concreting has been made difficult by the congestion of ducts or where there has not been proper compaction, resulting in large areas where spalling may allow aggressive agents to penetrate.[51]

During the workshop, Godart[3] also mentioned that "…When …external tendons…are in a wet atmosphere, generally due to bad construction details (lack of ventilation, absence of waterproofing, non tight inspection access…), wire failures caused by corrosion occur and the durability of the tendons is lowered."

From Japan, Mutsuyoshi[4] reported that internal post-tensioned concrete structures with cement grouting have been forbidden by the Japan Highway Public Corporation, because bad quality of grouting after construction has been found in many bridges. Now, almost all new prestressed concrete structures are being constructed using fully external tendons using transparent sheath with grouting.

2.2.2 *Experience in United States*

Out of the total number of bridges in the USA, approximately 20% are prestressed concrete bridges; only 3% of these were classified, in 1998, as structural deficient (bridges that can no longer sustain the loads for which they were designed).[43]

A comprehensive survey performed in 1999 by the American Segmental Bridge Institute (ASBI), found that concrete segmental bridges were performing well with time. Based on inspection reports using Federal Highway Administration (FHWA) guidelines, all segmental bridges were rated as "fair" or better. Of the 131 bridges, 99 percent had superstructure ratings of "satisfactory" or better, 79 percent had superstructure ratings of "good" or better, and 31 percent had superstructure ratings of "very good" or better.[52]

The first segmental bridge constructed in the US, in 1971, the John F. Kennedy Memorial Causeway near Corpus Christi, Texas, was inspected extensively in a Federal Highway Administration study in 1988, and no indications of distress or corrosion of the prestressing tendons were found.[33] This bridge was constructed using match-cast epoxy joints, as required by designers, considering the hot, humid, seawater environment of the Gulf of Mexico.

Recently, some tendon failures and corrosion related problems have come to light, especially in the state of Florida.[6,16] In 1999, one of the external tendons in the Niles Channel Bridge built in 1983, failed due to corrosion at an expansion joint. A 9-inch movement of the tendon through one of the deviation saddles was noticed first. When the tendon was removed for replacement, a void in the grout and heavy pitting in the prestressing strands inside the anchor head were found. (See Figures 2.5 and 2.6). In 2000, eleven tendons out of a total of 846 were replaced in the Mid-Bay Bridge built in 1993. Ten of the eleven tendons that were replaced were located at expansion joints. (See Figures 2.7, 2.8 and 2.9) Also, in 2000, several corroded tendons were discovered in segmental piers of the Sunshine Skyway Bridge, built in 1986, where the corrosion resulted from seawater in ducts, permeable concrete anchorage protection at the top of piers and splitting of polyethylene ducts.[6] (See Figures 2.10 and 2.11)

Figure 2.5 Plan View of Slipped Tendon at Deviation Saddle Niles Channel Bridge[16]

Figure 2.6 Advanced Corrosion of Strands within Anchorage Niles Channel Bridge[16]

Figure 2.7 The Mid-Bay Bridge, Florida[16]

Figure 2.8 Failure of Tendon 28-6 on the Mid-Bay Bridge[16]

Figure 2.9 Failure of Tendon 57-1 on the Mid-Bay Bridge at Expansion Joint Diaphragm[16]

Figure 2.10 The Sunshine Skyway Bridge, Tampa, Florida[16]

Figure 2.11 Tendon Corrosion inside the Sunshine Skyway Bridge Piers (Refer to Figure 2.3)[16]

In addition to the above, inspections in Florida bridges have revealed a large number of bleed water voids at anchorages, partially grouted tendons and ungrouted tendons, the same type of problems that were found in the Sidney Lanier Cable Stayed Bridge in Georgia and in the Boston Central Artery bridges.[6]

Freyermuth[6] has indicated that the major portion of the bridge tendon corrosion problems that have been observed in the U.S. have been identified with the following:[6]

1. An aggressive environment (northeast U.S. and Florida)
2. Areas with a low volume of post-tensioned construction
3. Contractors with no experience or expertise in post-tensioned construction
4. Grossly inadequate construction supervision
5. Design details without adequate provision for corrosion protection of tendons
6. Failures to respond to or correct construction problems

In particular, after analyzing the problems encountered in bridges located in the state of Florida, it appears that the tendon corrosion problems were due to:

1. Voids associated with accumulation of bleed water at tendons anchorages
2. Recharge of ungrouted tendon anchorages with salt water or surface drainage during construction.
3. Leakage through end anchorage protection details
4. Quality of the grout installation and grout materials
5. Splitting of polyethylene ducts
6. Deficiencies in implementation and inspection of grouting procedures.

The findings in Florida have lead to some immediate recommendations, with respect to the use of bonded post-tensioning systems:[53]

1. No precast concrete hollow column section should be specified below the waterline.
2. No PT tendons should be located in columns below the highest water splash zone elevation.
3. Grouting operation for vertical tendons should be carefully planned, tested and monitored. Stage and vacuum grouting should be specified in the upper section of tendons in combination with a pressurized sealed PT system and zero bleed grout.
4. Provide multiple levels of protection at anchorages, including permanent grout cap, epoxy material pour-back and polymer coating over the pour-back.
5. The impact of construction methods on the corrosion vulnerability of PT system should be thoroughly analyzed and designed for, especially for critical elements in aggressive corrosive environments.
6. PT redundancy system or practical replacement capabilities should be incorporated.
7. Corrosion detection methods should be included during the construction and service life of the structure.

After these important findings in Florida, many states are performing comprehensive investigations of their post-tensioned bridges, to determine the extent of the problem. These include among others the states of Texas, California, Virginia, and Georgia.[21]

In spite of the above, and as mentioned by Freyermuth,[6] the durability performance of prestressed and segmental post-tensioned bridges has been superior to all other types of construction. Recent improvements in grouting materials technology (anti-bleed thixotropic grouts), and training programs for grouting supervisors and inspectors are expected to yield significant results, reducing the incidence of corrosion problems in grouted tendons.

2.3 FACTORS AFFECTING THE DURABILITY OF BONDED POST-TENSIONED CONCRETE STRUCTURES

2.3.1 Exposure Conditions

There are four general environments where concrete structure durability may be a concern: coastal exposure, freezing exposure, and aggressive soils. These exposures may occur singly or in combination.

2.3.1.1 Coastal Exposure

Coastal exposures are one of the most severe environments for concrete structures. This is particularly true for structural components located directly in the seawater, as in the case of bridge substructures. Seawater contains dissolved salts which affect the durability of concrete. The most prevalent salts in order of quantity are sodium, magnesium and potassium chlorides and magnesium, calcium and potassium sulfates. These salts provide sources of chlorides and sulfates which can lead to corrosion of reinforcement and sulfate attack on concrete. To a lesser extent, these salts also provide a source of alkalis which may lead to expansive alkali-aggregate reactions if reactive aggregates are present. There are four main exposure zones for a structure in a coastal exposure: atmospheric zone, splash zone, tidal zone and submerged zone.[33]

Corrosion of steel reinforcement requires oxygen and thus generally occurs only in zones which experience some amount of drying. Also, corrosion rates are highest when humidity is in the 90-95% range. The greatest risk of corrosion occurs in the splash and atmospheric zones for these reasons. Corrosion in the tidal zone is normally limited due to the shorter drying periods and slower rate of oxygen diffusion through saturated concrete. The submerged zone of concrete has a low risk of corrosion due to lack of oxygen.[33]

Frost damage is most severe in concrete that is saturated, and therefore concrete within the tidal zone or immediately above the high tide level may experience the most significant damage. Freeze-thaw damage rarely occurs below the low tide level since the seawater would also have to freeze.[33]

Sulfate attack occurs primarily in zones where the concrete is submerged for some period, allowing greater sulfate concentrations. The greatest risk of sulfate attack normally occurs in the tidal zone and submerged zone. The same holds true for alkali-aggregate reactions due to alkalis in the seawater.[33]

The temperature range to which a structure is subjected also affects durability. Increases in temperature have an accelerating effect on many chemical reactions, including corrosion. The general rule of thumb is that a temperature increase of 10 degrees Celsius doubles the rate of reaction. Traditionally, seawater environments in cold climates, such as the North Sea, were viewed as the most severe exposure for structures. More recently, the accelerating effects of high temperatures have been recognized as equally or possible more severe than the combination of freezing temperatures and corrosive environments.[33]

2.3.1.2 Freezing Exposure

Environments where structures may be exposed to freezing temperatures may lead to freeze-thaw damage of concrete. A secondary effect is that the use of deicing chemicals in freezing exposures can exacerbate freeze-thaw damage and may lead to corrosion of steel reinforcement if the deicing agents contain chlorides.[33]

The severity of freeze-thaw damage of concrete is a function of the presence of moisture in the concrete and the number of times the moisture freezes and thaws. Frost damage worsens when repeated cycles of freezing and thawing occur. Thus a moderate winter climate which experiences many freeze-thaw cycles can cause more frost damage than a severe winter climate that remains below freezing for long periods.[33]

2.3.1.3 Aggressive Soils

Chemical attack on concrete in the form of sulfate attack or alkali-aggregate reactions may occur in soils containing sulfates or alkalis. The presence of these aggressive agents must be accompanied by moisture for attack to occur (assuming the concrete is susceptible to either form of attack). Moisture provides the transport mechanism for sulfates and alkalis to penetrate concrete, and is also necessary for the deleterious reactions to occur.[33]

2.3.2 *Concrete Durability*

Concrete is typically the first level of protection for the reinforcement. Extensive research has been performed on the many subject areas pertaining to concrete durability.[26] Several aspects related to concrete durability are discussed in this section, but a detailed literature review on the subject can be found in West.[33]

2.3.2.1 Sulfate Attack

Sulfate attack is a fairly intricate process that causes cracking of the concrete. Sulfates react to the C_3A in concrete to form ettringite. The ettringite occupies a much larger volume in the concrete that then causes cracking due to expansive stresses. Sulfate attack can be controlled by utilizing cements with low C3A contents, such as Type V cements. Pozzolan addition has also been found to be helpful in preventing sulfate attack. (from various references in Schokker[34])

2.3.2.2 Freezing and Thawing Damage

Saturated or nearly saturated concrete can be susceptible to freeze-thaw damage due to the expansion of water during freezing. Fortunately, this problem can be avoided by the addition of an air-entraining admixture. (from various references in Schokker[34])

2.3.2.3 Alkali-Aggregate Reaction

Alkali-aggregate reactions are chemical reactions between alkalis and certain types of aggregates. The reactions may be alkali-silica or alkali-carbonate, depending on the type of aggregate. The alkali-silica reactions produce an alkali-silica gel that possesses expansive properties and can lead to cracking and deterioration of the concrete. The reaction of alkalis with carbonate aggregates is referred to as the dedolomitazation of the aggregate. The alkali-carbonate reaction products are prone to swelling in a manner similar to clays.[33]

2.3.2.4 Carbonation

Carbonation of concrete occurs when atmospheric carbon dioxide penetrates the concrete. In the presence of moisture, carbon dioxide will react with calcium hydroxide in the concrete to produce calcium carbonate.

The formation of calcium carbonate reduces the pH of the concrete to as low as 8, where the passive film is no longer stable allowing corrosion to begin.

The process of carbonation is slow in good quality concrete. The rate of carbon dioxide penetration is a function of the square-root of the exposure time. Factors affecting the rate of carbonation include the concrete permeability, cracking, the moisture content of the concrete and relative humidity. The presence of cracks will allow the carbonation front to reach the steel rapidly (a local scale). This may contribute to the formation of macrocell corrosion at the crack. Carbonation will not occur in concrete that is saturated or very dry. The rate of carbonation is highest for relative humidity of 50% to 70%.

Carbonation may be slowed by specifying concrete with low permeability. Options include the use of low water-cement reactions, use of superplasticizers to reduce water demand, and the use of mineral admixtures. Compaction and proper curing are needed to ensure low permeability. Concrete surface treatments or sealers may slow penetration of carbon dioxide. (from various references in West[33])

2.3.2.5 Cracking

The effect of cracking on the corrosion of reinforcing or prestressing steel is controversial. One viewpoint is that cracks reduce the service life of structures by permitting rapid penetration of carbonation and by providing a means of access of chloride ions, moisture, and oxygen to the reinforcing steel. Thus, cracks accelerate the onset of corrosion.[54]

The other viewpoint is that while cracks accelerate the onset of corrosion, that corrosion is localized. With time, chlorides and water penetrate even uncracked concrete and initiate more widespread corrosion. Consequently, after a few years of service for concrete with moderate to high permeability there is little difference between the amount of corrosion in cracked and uncracked concrete.[54]

With respect to cracking, Qing Li.[55] concluded that "…Load-induced cracking does affect the corrosion initiation of the reinforcement in structural concrete, … From the test results, it is almost certain that the corrosion initiates at the cracked sections of reinforced concrete structural members. How this localized corrosion propagates, leading to further longitudinal surface cracking, delamination, debonding , and the ultimate capacity reduction in terms of strength and stiffness of the structure, is of consequential importance to structures in marine environment, where an ample supply of saltwater and oxygen is available. In particular, cyclic tides and/or waves pump saltwater in and out of the cracks, significantly exacerbating structural capacity deterioration. This intricate phenomenon has not been examined thoroughly thus far and needs considerable attention from the research community."

An in-depth analysis of cracking and corrosion is included in Reference 33.

2.3.3 Grouts for Post-Tensioning

As summarized in West,[33] cement grout bonds the post-tensioning tendon to the surrounding concrete and provides corrosion protection for the tendon. Corrosion protection is in the form of a barrier to moisture and chloride penetration, and the presence of an alkaline environment for the tendon.

An optimum grout for post-tensioning combines desirable fresh properties with good corrosion protection. The fresh properties of the grout influence how well the grout fills the duct. The corrosion protection provided by the grout is rendered less effective if the duct is only partially or intermittently filled with grout. The presence of voids or discontinuous grouting may also permit movement of moisture and chlorides along the length of the tendon. Important grout fresh properties include: fluidity, bleed resistance, volume change, set time.

The fresh properties of grout can be controlled through water-cement ratio, the use of chemical and mineral admixtures, and by the type of cement.

Admixtures include the use of anti-bleed agents sometimes referred to as thixotropic admixtures. This class of admixture gives the grout gel-like properties to minimize bleeding, while permitting the grout to become fluid when agitated (mixed, pumped, etc.).

Grouting techniques are critical. As reported by Hamada, et al.[27], at the Ghent Workshop, …it has been found in Japan … that serious deterioration has occurred in prestressed bridges due to insufficient grouting."

2.3.3.1 Voids and Bleed Water

Voids can be formed in the post-tensioning duct from incomplete grouting, trapped air pockets, or from the evaporation of bleed water pockets. Top quality grout is of little benefit if poor grouting procedures result in large void formations, which provide no protection to the strand and no transfer of bond. Proper venting of the post-tensioning duct is critical for complete grouting. The void between the tendon and the post-tensioning duct is a very complex space. For instance, a parabolic shaped duct with a tensioned tendon may have a number of small voids of varying shapes and sizes, and very stiff grout may not fill the interstices.[56]

Bleed lenses form as a result of the separation of water from the cement. This sedimentation process is accentuated by the addition of seven –wire strand, which acts as a "water-transport mechanism."[57]

The spaces within the individual twisted wires that form the strand are large enough to allow easy passage of water but not cement. Ducts with vertical rises will typically cause more bleed due to the increased pressure within the grout column. Intermediate bleed water lenses may form in tall vertical ducts, leaving a void through the cross-section of the duct exposing the tendon. Even in parabolic draped ducts, any bleed water will tend to gather near the highest intermediate points, leaving voids in the duct.[34]

Grouts containing anti-bleed admixture, or thixotropic grouts, can be bleed resistant even when used in ducts with large vertical rises.[44] These grouts are able to retain their water even under high pressures and can eliminate significant void formation when proper grouting procedures are followed.[34]

From the Florida experience, Pielstick[59] reported that "…the corrosion [of prestressing steel] resulted from the absence of grout due to accumulated bleed water at the anchorages leaving voids. The bleed water was either reabsorbed into the grout or evaporated. It appears that additional corrosion resulted when these voids were recharged with water and air leaking through at the anchorages. ….Multiple voids were …found near the anchorages which were then filled with grout by the vacuum injection method. This vacuum injection method removes as much air as possible and measures the air volume, then replaces the measured air volume with grout."

2.3.3.2 Grout Cracking

Grouts are injected after the concrete elements have been post-tensioned, and therefore there is no compression force in the hardened grout, making the grout matrix vulnerable to cracking.

The effect of grout cracking on prestressing steel corrosion has not received enough attention by researchers. Hamilton[40] reported after his series of accelerated corrosion tests, that "…the outline of the crack…is in line with the most intense area of corrosion. Away from the crack location the intensity of the pitting corrosion decreases. In some areas a greenish-white corrosion was noted which was still moist when the grout was removed. Usually within a day the corrosion product would dry and the remaining deposit would be red or black."

2.3.4 Corrosion of Steel Components in Post-Tensioning Systems

Corrosion of prestressing steel is generally of greater concern than corrosion of passive mild steel reinforcement because of the possibility that corrosion may cause a local reduction in cross section and failure of the steel. The high stresses in the steel also render it more vulnerable to stress corrosion cracking and, where the loading is cyclic, to corrosion fatigue.[54]

Corrosion of prestressing reinforcement can be divided into three main types: conventional or electrolytic corrosion (rust, small pits, etc), stress corrosion, and hydrogen embrittlement corrosion. Conventional corrosion is by far the most widespread form.[3]

2.3.4.1 Corrosion Fundamentals

From Schokker[34], before considering the corrosion of prestressing steel in concrete, a general corrosion theory for metals must be understood. Corrosion of iron is an electrochemical process governed by Equations 2-1 and 2-2, commonly known as half-cell reactions:

$$\mathbf{Fe \rightarrow Fe^{2+} + 2e^-} \qquad \text{(anodic half-cell reaction)} \qquad (2\text{-}1)$$

$$\mathbf{2H_2O + O_2 + 4e^- \rightarrow 4OH^-} \qquad \text{(cathodic half-cell reaction)} \qquad (2\text{-}2)$$

The anode (where electrochemical oxidation takes place) and the cathode (where reduction takes place) form on the metal surface. Iron is oxidized into ferrous ions at the anode as shown in Equation 2-1. The ferrous ions are converted to $2Fe(OH)_3$ (commonly known as rust) through a number of reactions. A summary of rust formation is shown in Equations 2-3 and 2-4.[60]

$$\mathbf{2Fe + 2H_2O + O_2 \rightarrow 2Fe^{2+} + 4OH^- \rightarrow 2Fe(OH)_2} \qquad (2\text{-}3)$$

which can further react to give:

$$2Fe(OH)_2 + H_2O + \frac{1}{2} O_2 \rightarrow 2Fe(OH)_3 \quad \text{(Rust)} \qquad (2\text{-}4)$$

The anodic and cathodic areas are regions of different electrochemical potential that develop due to two different metals (which therefore have different potentials) or a single metal with surface differences (metallurgical or local variations in electrolyte).[61] The anode and cathode locations can change often and have an irregular pattern leading to a somewhat uniform corrosion or the locations can be more fixed and localized.

Passivity

Steel is an active-passive metal, and therefore its corrosion rate depends on potential as shown in Figure 2.12. Under typical conditions, steel in concrete is in a passive state and a passive protective film is found on the steel surface. Chlorides (and lowered pH) in the surrounding concrete have been shown to cause a breakdown of the passive film at potentials that should be well within the passive region.[62]

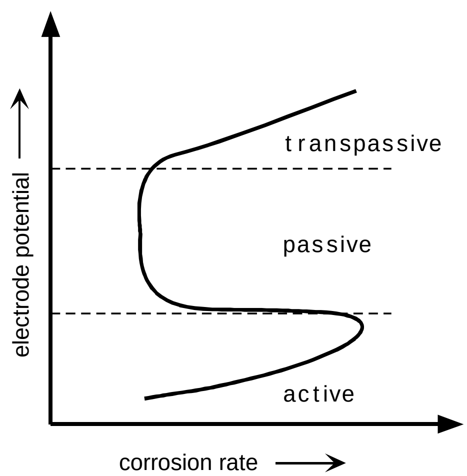


Figure 2.12 Passive-Active Behavior in Steel[34]

Corrosion of Steel in Concrete

In the case of corrosion of steel in concrete, the anodes and cathodes are formed on the steel surface with the cement paste pore solution acting as an electrolyte. Figure 2.13 shows the basic corrosion process for steel in concrete. The rust product occupies a much larger volume than the products that go into its formation which can cause splitting tensile stresses in the concrete.[34]

Half-Cell Potential Measurements

Electrical potential measurements are useful to monitor the corrosion of the steel embedded in the concrete. Electrical half-cell potential measurements are often used for this purpose. Because it is impossible to measure the absolute value of a half-cell potential, measurements must be made of two half-cell potentials with one used as a reference potential value. For convenience, the hydrogen half-cell reaction at standard state is arbitrarily defined as having a potential value of +0. Potentials may be reported in terms of the primary reference electrode, known as the Standard Hydrogen Electrode (SHE) or directly in terms of the secondary reference electrode used during half-cell potential measurements. For half-cell potentials measured in concrete specimens, common secondary reference electrodes include the Copper-Copper Sulfate Electrode (CSE) and the Saturated Calomel Electrode (SCE). Table 2.4 gives the potential versus SHE for these reference electrodes.[60] Half-cell potentials throughout this document will be reported as millivolts versus SCE. In order to change these values to compare with results from the other common reference electrode, CSE, simply add +77 millivolts to the values.[34]

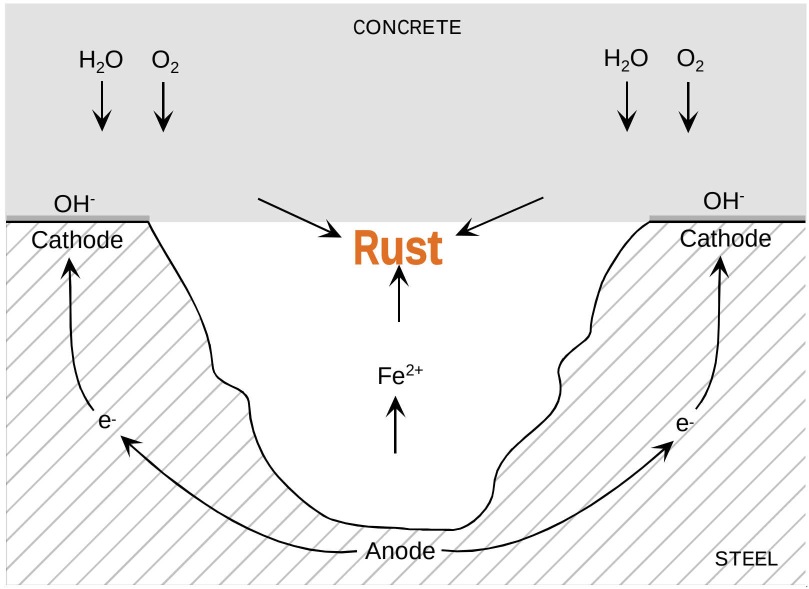


Figure 2.13 Corrosion of Steel in Concrete[34]

Table 2.4 Common Reference Electrode Potentials versus SHE[60]

	Half-Cell Reaction	***Potential (V vs. SHE)***
Copper-Copper Sulfate (CSE)	$CuSO_4 + 2e^- = Cu + SO_4^{2-}$	+0.318
Saturated Calomel Electrode (SCE)	$Hg_2Cl_2 + 2e^- = 2Hg + 2Cl^-$	+0.241
Standard Hydrogen Electrode (SHE)	$2H + 2e^- = H_2$	+0.000

2.3.4.2 Chloride Induced Corrosion

"Chloride attack has been found to be the severest factor in Japan and many prestressed concrete bridges have been found deteriorated due to this factor. Thus, it is supposed that chloride attack will be the most important factor in future durability of prestressed concrete bridges in Japan."[4]

"…The literature investigation revealed that almost all deterioration cases were caused by chloride induced corrosion."[27]

Chloride induced corrosion is the most common form of corrosion in reinforced concrete. This type of corrosion can be relatively quick and can localize to cause significant reductions in cross-sectional area. The rust product causes cracking and spalling as well as unsightly staining at the concrete surface.

The role of chlorides in depassivation has been much debated. The general consensus is that once the level of chlorides in the concrete at the steel exceeds a certain limit, the passive film either breaks down or is no longer able to protect the steel from corrosion. In general terms, the role of chlorides in depassivation may take one or a combination of the following forms:[33]

- chloride ions may disperse the passive film.

- chloride ions may make the film permeable to Fe^{2+} ions allowing the anodic reaction to occur even when the passive film is present.
- chloride ions may penetrate the passive film and anodic reactions with Cl^- acting as a catalyst may occur.
- chloride ions may reduce the pH, making the passive film unstable.

Chloride thresholds for corrosion are controversial because corrosion is dependent on so many variables:[61]

- proportioning of concrete
- type and specific area of cement
- water-cement ratio
- sulfate content
- curing conditions, age, environment
- carbonation
- temperature and relative humidity
- condition of reinforcement

Chlorides may be present in the concrete from any of the concrete constituents. They may also penetrate the concrete from external sources, mostly commonly de-icing chemicals, which combine with the melted snow or ice and often run down the surface of the superstructure and substructure elements; and, seawater, from immersion or salt-water spray.[33,34]

Three different analytical values have been used to designate the chloride content of fresh concrete, hardened concrete, or any of the concrete mixture ingredients: (a) total, (b) acid-soluble, and (c) water-soluble.[54] Work at Federal Highway Administration Laboratories and after field bridge deck studies in California and New York, demonstrated that for hardened concrete subject to externally applied chlorides, the corrosion threshold was 0.2 percent acid-soluble chlorides per weight of cement[54] (around 0.6 to 0.9 kg of Cl^-/m^3 or 1.5 lb/yd^3). A detailed discussion on chloride threshold values is included in Reference 33, pages 39-41.

To define the service life of a structure based on chloride exposure, two concepts need to be defined: the initiation period (time to the onset of corrosion) and the propagation period.

The initiation period t_i defines the time it takes for chlorides to penetrate the concrete cover and accumulate at the location of the embedded steel in a sufficient quantity to break down the protective passive layer on the steel, initiating an active state of corrosion. The length of this period is a function of the concrete quality, depth of cover, exposure conditions (including the level of chloride at the surface and the temperature of the environment), and the threshold chloride concentration required to initiate corrosion. A simple approach used to predict the initiation period is to assume that ionic diffusion is the mechanism of chloride transport and to solve Fick's second law of diffusion.[63]

The propagation period t_p defines the time necessary for sufficient corrosion to occur to cause an unacceptable level of damage to the concrete structure or element under consideration. The length of this period depends not only on the rate of the corrosion process, but also on the definition of "unacceptable damage." This level of damage will vary depending on the requirements of the owner and the nature of the structure.[63]

<u>Uniform Corrosion</u>

Uniform corrosion is generalized corrosion over a large area. This type of corrosion may have a large amount of metal loss and cause cracking and staining over large areas. Since the damage is not localized, catastrophic failure is least likely for this type of corrosion than for the other types of corrosion found in prestressed concrete structures.[34]

Pitting Corrosion

Pitting corrosion is very common in reinforced concrete structures and can be quite destructive due to the concentrated pits of corrosion causing large reductions in cross-sectional area with a small amount of overall metal loss. The pitting process is a self-propagating process.[62] As the pits grow, the surface can be undercut resulting in a deceivingly large pit and making detection difficult even on bare steel. A schematic of the process for iron is shown in Figure 2.14.[34]

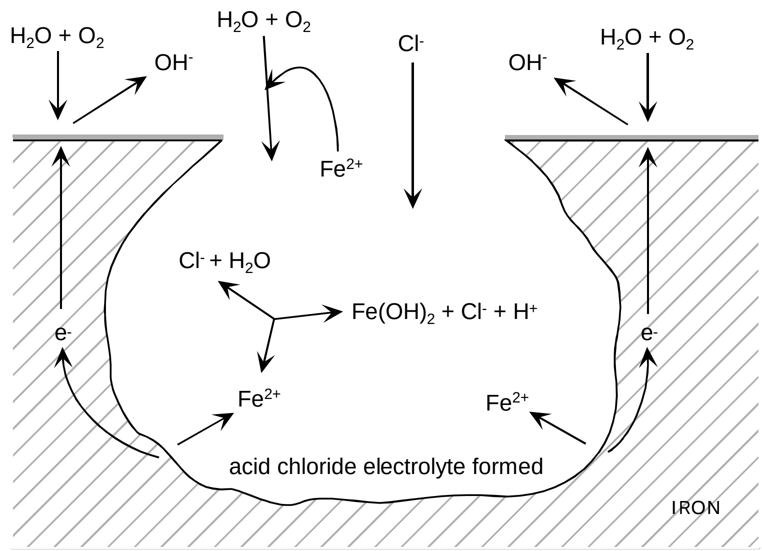


Figure 2.14 The Self-Propagating Process of Pitting Corrosion[34]

2.3.4.3 Hydrogen Embrittlement and Stress Corrosion Cracking

"…there are countries who do not permit the use of galvanized duct because of the fear that galvanizing could cause hydrogen embrittlement of the prestressing steel. Even though widely used there is no evidence known to the author [Ganz] where the use of galvanized duct would have caused damage to tendons made of cold drawn wire and 7-wire strand."[64]

"….its prestressing [of the Saint-Cloud Viaduct] is completely internal to the concrete. ….it was strengthened in 1979 by an additional external prestressing which is deviated in a vertical plane…In 1998, one of the additional tendons which were in the Northern side cell and which had a 300 m length broke in its middle…the prestressing wires were sensitive to the stress corrosion, and the majority of wires presented this type of cracking."[3]

As described by Schokker,[34] the combination of tensile stress and a corrosive environment can cause susceptibility in steel to certain types of environmentally induced cracking. The types of environmentally induced cracking most often associated with prestressing steel are stress-corrosion cracking and hydrogen embrittlement. Both types of corrosion may lead to brittle failures with minimal metal loss in the affected area. The most dangerous aspect of hydrogen embrittlement and stress-corrosion cracking failures is the potential for structural collapse without warning.

Jones[60] defines stress corrosion cracking (SCC) as brittle failure of an alloy in a corrosive environment at relatively low tensile stress. Hydrogen embrittlement (HE) is defined as brittle fracture caused by the absorption of atomic hydrogen into a metal alloy.

Schupack and Suarez[65] performed a survey in 1982 to investigate corrosion incidents in prestressed structures during the previous five years. The study indicated a small number of incidents with only 1/5 of these reported as possible SCC / HE brittle failures. Most of these incidents were found in unbonded post-tensioned construction. The failures reported were within an isolated area in the structure and were not catastrophic collapses.

Stress corrosion cracking and hydrogen embrittlement both cause brittle fractures in a corrosive environment under tensile stress, but one difference is their behavior in the presence of cathodic protection. Cathodic protection can suppress stress corrosion cracking, but tends to enhance hydrogen embrittlement.[60,66] For this reason, cathodic protection can be problematic for prestressed structures.

The effect of galvanizing on hydrogen embrittlement of prestressing wire is a topic of concern. The zinc coating on the galvanized wire reacts with the alkaline Portland cement in fresh concrete or grout and evolves hydrogen:[67]

$$\mathbf{Ca(OH)_2 + Zn + 2H_2O = Ca(Zn(OH)_4) + H_2} \qquad (2\text{-}5)$$

A study by Yamaoka, Hideyoshi and Kurachi[67] found indications that galvanization of prestressing steel helps protect the steel from hydrogen embrittlement.

2.3.4.4 Steel Geometry and Crevice Corrosion

As stated by Schokker,[34] the cross-section of prestressing strand makes it more susceptible to corrosion than traditional steel reinforcement. The strand is made up of a twisted bundle of seven individual wires, which give it a higher ratio of surface area to total cross-sectional area than we would find for a solid bar with the same diameter as the strand. The contact between the wires can also cause corrosion at the interfaces (crevice corrosion).

The geometric constraints of the crevice enhance the formation of chloride ion concentration cells. Once corrosion has initiated, it progresses similar to pitting corrosion. Due to the geometry of the crevice, Fe2+ ions can not disperse easily, and chloride ions are drawn into the crevice by the positive charge accumulation. The process becomes autocatalytic as the presence of chloride ions leads to formation of hydrochloric acid (HCl) and higher corrosion rates develop.[33]

2.3.5 Anchorage Sealing

Anchorage corrosion may lead to cracking and spalling of the concrete in the vicinity and even failure of the anchorage. Corrosion of the anchorage and strand stubs may also allow moisture entry into the duct and subsequent tendon corrosion. Not all multistrand post-tensioning systems include an end cap. Anchorages are commonly recessed in a pocket at the end or edge of the concrete element. Corrosion protection for the anchorage normally consists of filling the anchorage recess or pocket with mortar or concrete. The location of the anchorage within the structure can also play a role in the onset of corrosion and corrosion development. In many structures, the anchorages are located at the ends of structural elements below expansion joints, or at exterior member ends or slab edges. These locations are prone to concentrated exposure with moisture and chlorides, and often lead to severe anchorage corrosion damage.[33]

2.3.6 Precast Segmental Construction – Joint Performance

Dry joints or poorly sealed segmental joints have been shown to be extremely detrimental to the durability of post-tensioned structures, either with internal[2,22] or external post-tensioning.[16] Experiences in the United Kingdom[2,22] led to the ban in 1992 on the use of internal bonded post-tensioned structures with discontinuous (poorly sealed) ducts. The general experience with epoxy joints in the United States has been much more favorable.

2.4 CORROSION PROTECTION OF BONDED POST-TENSIONED CONCRETE STRUCTURES

"…The objective of the corrosion protection must be to achieve a design life of the tendons which is comparable to that of the structure in which they are placed."[31]

"Without exceptions, and regardless of cost, improvements in grouting practice and improvements in the quality of global tendon corrosion protection details, must be implemented to provide full assurance of the long-term durability (100+ years) of post-tensioning tendons"[21]

Corrosion protection of bonded post-tensioned concrete structures can be achieved by the careful design of the multilayer protection system. As shown in Figure 2.15, protection measures include surface treatments on the concrete, the concrete itself, the duct, the grout and strand or bar coatings such as epoxy or galvanizing. Post-tensioning also provides the opportunity to electrically isolate the prestressing system from the rest of the structure.

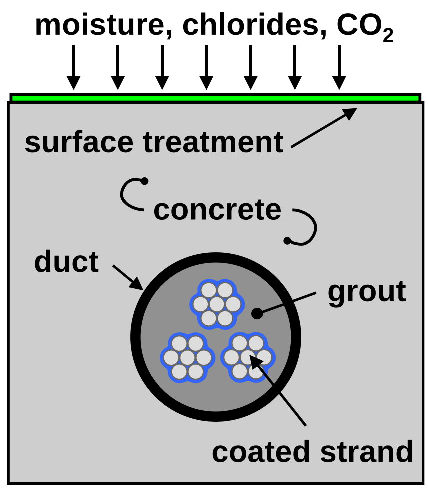


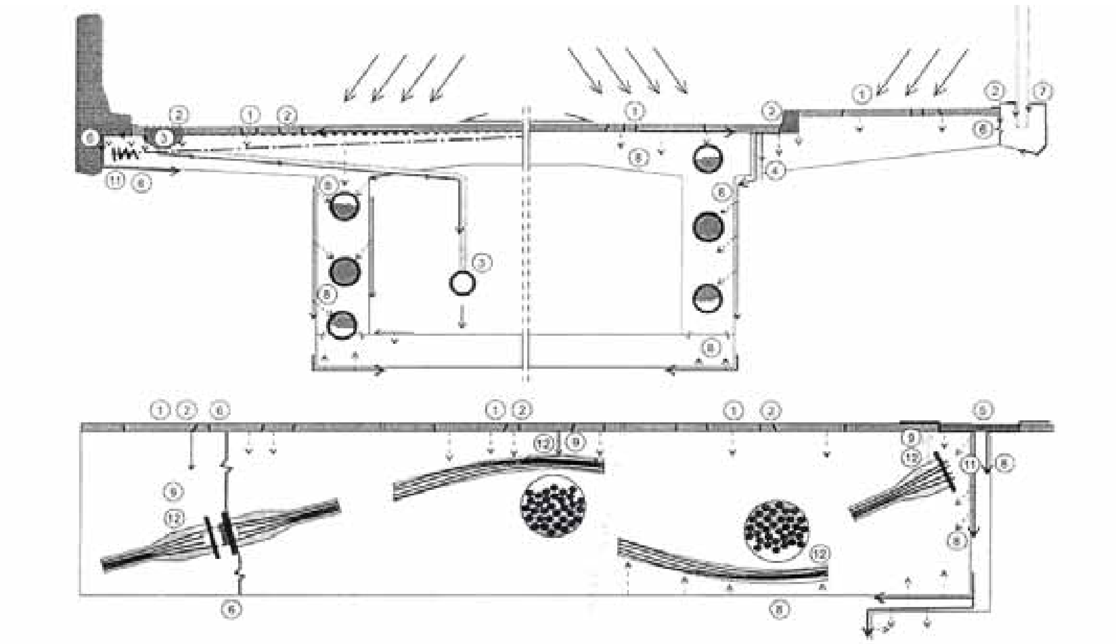
Figure 2.15 Multilayer Corrosion Protection for Bonded Post-Tensioned Systems[33]

As described by Matt[68] weak links could exist in every layer. In the first layer, water can penetrate through defective or inadequate design of non-structural elements such as waterproofing membranes, drainage systems, etc. and reach the concrete structure. In a second layer, water can be transported through the concrete cover by diffusion and capillary absorption. The rate is determined by the quality of the concrete (thickness, permeability, cracks, honey-combing, etc.) The third barrier/layer consists of the corrosion protection system of the post-tensioning tendons themselves. It can be impaired by open grouting vents, leaking ducts, cracked and porous pocket concrete as well as grout voids, etc. In the fourth layer the longitudinal water transport in an inclined tendon has to be considered in case the grouting may not be perfect. In the fifth layer, the prestressing steel can also corrode at locations remote from the point of ingress. Figure 2.16 shows the possible hazard scenarios in a typical box girder segment.

To ensure better corrosion protection of prestressing steel, Schupack[69] indicates that the following should be considered, particularly if the environment of use is aggressive or control of construction is doubtful:

- Designs should minimize concrete cracking, particularly in aggressive environments;
- High-quality, low – permeability concrete;
- Construction techniques should ensure specified concrete cover to all reinforcement;
- Tough plastic sheaths for post-tensioning;
- Improved grouting methods and materials (that is, nonbleeding grout);
- Use of electrically isolated tendons;

- New tough polymer coatings for strands and bars;
- Stainless steel-clad wires and bars;
- Stainless steel (now more than 10 times as expensive as A 416 strands);
- Low-carbon alloy bars with good stress-corrosion-cracking resistance;
- A more inherently corrosion-resistant steel; and,
- High-strength bars with subtle deformations that can replace strand.



Hazard scenarios for prestressing steel in a typical box girder bridge: Indication of potentially "weak points" where water (possibly contaminated with chlorides) can gain access to the tendons and cause corrosion

Non-structural elements:

1. *Defective wearing course (e.g. cracks)*
2. *Missing or defective waterproofing membrane incl. edge areas*
3. *Defective drainage intakes and pipes*
4. *Wrongly placed outlets for the drainage of wearing course and waterproofing*
5. *Leaking expansion joints*
6. *Cracked and leaking construction or element joints*
7. *Inserts (e.g. for electricity)*

Corrosion protection system:

8. *Defective concrete cover*
9. *Partly or fully open grouting-in- and outlets (vents)*
10. *Leaking, damaged metallic ducts mechanically or by corrosion*
11. *Cracked and porous pocket concrete*
12. *Grout voids at tendon high points*

Figure 2.16 Hazard Scenarios[68]

2.4.1 Structural Form

A detailed description of corrosion protection variables is included in West.[33] A summary of the most important aspects are included here as a reference.

2.4.1.1 Drainage

Since the most common cause of reinforcement corrosion is moisture-borne chlorides, whether as seawater or de-icing chemicals, adequate drainage is a critical factor in the selection of structural form. Adequate superstructure drainage must be provided, and must ensure chloride-laden water does not come in contact with the substructure.[33] Use of impermeable deck sealants or membranes should be considered.

2.4.1.2 Joints

A common source of substructure corrosion problems is moisture and chlorides dripping onto substructure components through leaking deck joints. Proper joint design and maintenance is required when severe conditions are encountered.[33]

2.4.1.3 Splashing

Substructure components adjacent to roadways where de-icing chemicals are used can be prone to corrosion damage due to splashing. Increasing the distance between the roadway and substructure may increase initial construction costs, but may reduce long term costs by avoiding this type of corrosion damage.[33]

2.4.1.4 Geometry

Other aspects of the structural geometry can influence durability. Decreasing the exposed surface by an efficient design and increasing concrete cover to safe limits can decrease the corrosion potential.

2.4.2 Structural Design Details

2.4.2.1 Cracking

Concrete cracking provides easy access for moisture and chlorides to reach the reinforcement. Although the significance of cracking on corrosion is often debated, cracking should be avoided or minimized where possible.[33]

2.4.2.2 Reinforcement Detailing

Reinforcement detailing not only affects cracking in the structure, it also can influence construction. Congested reinforcement details make concrete placement difficult and can lead to poor compaction and voids.[33]

2.4.3 Surface Treatments

Concrete surface treatments work to improve corrosion protection by preventing moisture and chlorides from entering the concrete. Surface treatments include waterproof membranes and surface polymer impregnation. In the latter, the exposed surface of the concrete is impregnated with polymer that fills the voids and cracks in the concrete, providing a barrier with very low permeability. Various overlays may be used to provide a low permeability barrier over existing concrete. Options include polymer concrete overlays, latex-modified concrete overlays and overlays with low permeability Portland cement concrete.[33]

2.4.4 Concrete as Corrosion Protection

Concrete acts as a physical barrier to moisture and chlorides, and provides the alkaline environment necessary for formation of the passive film on the surface of steel.[33]

2.4.4.1 Concrete Permeability

The permeability of concrete controls the rate at which moisture, oxygen and carbon dioxide penetrates the concrete. Because the penetration of moisture provides the transport mechanism for chlorides and other aggressive substances, lowering the concrete permeability increases the length of time before aggressive agents reach the steel, and thus improves corrosion protection. The permeability of concrete is affected by four general factors:[70,71] pore structure of the cement paste, aggregate, voids in the concrete, and, cracking of the concrete.

A detailed description of each factor is included in Reference 33. The discussion includes the used and effects of low water/cementitious material ratio and the use of supplementary cementitious materials.

High Performance Concrete (HCP) as used in the text, refers to low permeability concrete with improved strength f'c=10000 psi, w/c = 0.29, 25% replacement by fly ash, and superplasticizer to reach a slump of 8 inches.

2.4.4.2 Cover Thickness

The thickness of concrete cover over the reinforcement plays a significant role in corrosion protection. Increased clear cover provides improved protection for the steel, particularly if low permeability concrete is used. The penetration of chlorides over time can be approximated by a square-root time law.[72] This means if the concrete cover thickness is doubled, it will take approximately four times as long for chlorides to penetrate to the depth of the reinforcement.[33]

2.4.4.3 Corrosion Inhibitors

Many types of corrosion inhibitors are on the market today. They are intended to slow the corrosion process of steel in concrete without adversely affecting other properties and are typically included as an admixture in the fresh concrete. Corrosion inhibitors can be divided into three basic types by the method in which they slow the corrosion process.[73,74]

Anodic Inhibitors (Passive System)

Anodic inhibitors react with the steel to form a protective film, and proper dosage depends on the amount of chlorides penetrating the concrete. If the amount of chlorides is too high for the dosage of corrosion inhibitor, then all of the anodic sites are not eliminated and corrosion continues at a rate greater or equal to that of untreated concrete. The popular corrosion inhibitor, Calcium Nitrite, is an anodic inhibitor.

Cathodic Inhibitors (Active System)

Cathodic inhibitors form a barrier around the cathodic site to reduce chloride ingress. These inhibitors tend to be less efficient than anodic inhibitors. Silica fume is an example of a cathodic inhibitor.

Mixed Inhibitors (Passive-Active System)

Mixed inhibitors combine the inhibiting traits of both the anodic and cathodic type inhibitors.

2.4.5 High Performance Grouts

As West[33] has stated, the selection of suitable grout proportions and admixtures requires careful consideration of the grout fresh properties and corrosion protection. The effects of various admixtures and grout proportions on fresh properties and corrosion protection have been studied by several researchers.[40,75] Schokker[34] performed an extensive research study to develop two optimized grouts for post-tensioned. The study was part of this research project.

Additional information on mix proportioning and guide specifications for grouts for post-tensioning is provided by the PTI "Guide Specification for Grouting of Post-Tensioned Structures,"[19] the fib report "Grouting of Tendons in Prestressed Concrete,"[26] and the VSL report "Grouting of Post-Tensioning Tendons."[23]

As a response for the need of better grout mixes, new prepacked grouts are available with thixotropic properties. One of these grouts is in the form of a plastic gel when at rest, and is instantly fluidized when it is shaken. This grout uses a w/c = 0.35, and two liquid additives: a stabilizer that retards hydration of the cement and fluidizes it, and also reduces shrinkage; and, a set activator that also acts as a thixotropic agent.[76]

In an effort to provide a more consistent grout material, the Florida Department of Transportation is requiring that pre-bagged resistant grouts be used. The ASBI and PTI have recommended anti-bleed or low bleed grouts meeting a series of performance requirements. These grouts reduce the size and number of voids due to bleed water, but anchorages should still be probed or visually inspected when the grout has set.[59]

2.4.6 Ducts for Post-Tensioning

Typically galvanized corrugated metal ducts have been used in Post-Tensioning applications. However, as a consequence of the corrosion concern, specific corrugated plastic duct systems for bonded post-tensioning have been introduced on the market, since the 1990's.[31] To date, many designers do not considered it necessary to always require plastic ducting to provide and extra barrier.[77] However, in view of the substantial corrosion found with galvanized ducts in the exposure tests of the current project, it seems like a prudent step to use plastic duct in any application that has even a moderate risk of exposure to chlorides.

Plastic duct systems, when combined with suitable accessories such as connection details and anchorage caps, provide a complete encapsulation of the post-tensioning tendon. Considering that not all plastic ducts are suitable for post-tensioning applications, *fib* has taken the initiative some years ago, and has prepared a technical report for the testing and approval of plastic duct systems for use in post-tensioning.[25]

To avoid excessive wobble of the tendons, excessive friction losses during stressing, and leakage of grout during injection, more "robust" ducts of durable polyethylene are being introduced.[25,64]

Some of these plastic duct systems have now been used with excellent experience. Plastic duct systems had been specified exclusively in the UK since 1996, and are now being specified also in Florida and in Switzerland for the general application in aggressive environment. Experience has shown that plastic ducts made of polypropylene (PP) perform better in warm climates than those made of polyethylene. It has also been found that the plastic ducts need to be protected from large local transverse loads at supports near the tendon profile high points by the provision of sufficiently rigid half shells.[31]

Changes in the material requirements for the High-Density Polyethylene (HDPE) duct systems have been suggested for all external post-tensioning systems. Robust plastic ducts have been recommended for internal tendons meeting the requirements of the fib Technical Report[25] as referenced in the PTI Guide Specification for Grouting of Post-Tensioned Structures.[19,59]

2.4.7 Coatings on Prestressing Steel

2.4.7.1 Metallically coated prestressing steels

As indicated by Ganz[31] zinc coated prestressing steel is rarely used for post-tensioning. Zinc provides a sacrificial protection of the prestressing steel. The durability of the protection by zinc depends primarily on the consumption rate of zinc in the actual environment, and the available thickness of the zinc layer. Zinc coating is relatively insensitive to local damage and the prestressing steel remains protected even if the zinc coating is locally damaged. Galvanized prestressing steels have been standardized in the French standard NF A35035. It is understood that a European standard and an ISO standard are being prepared on galvanized prestressing steels.

2.4.7.2 Non Metallic Coatings for Prestressing Steels

From Ganz,[31] the most commonly used non-metallic coating is epoxy resin. Experience has shown that epoxy coated strand needs to have the interstices between wires completely filled with epoxy to avoid migration of water / humidity along the strand.[31] Further issues in ongoing discussions are on the effect of local defects in the coating in an aggressive environment such as under chloride attack. It has been reported that local defects may lead to accelerated pitting corrosion at these locations. The thickness of coating needs to be carefully controlled for reliable anchorage by wedges.[64]

Other non-metallic coatings have been developed including resins with delayed curing (After-Bond), tar epoxy resins, and others. However, experience is still limited for applications in pretensioning and post-tensioning.[31]

While there is no dispute that epoxy coating will extend the time to corrosion damage, compared with uncoated steel, the long-term performance remains somewhat uncertain. Not all the factors affecting corrosion performance are understood, and there are many examples of good performance, as well as examples of premature corrosion damage. The dominant factors affecting performance are the number and size of defects in the coating and the long-term adhesion of the coating to the steel.[78]

2.4.8 Prestressing Steel – Other Prestressing Materials

Fiber-reinforcement polymer (FRP), stainless steel, stainless-clad steel, and MMFX steel bars comprise the new generation of concrete reinforcing materials being used for durable construction.[79]

With respect to FRP reinforcement, Clemena states that: "…Until we learn more …, I would consider the corrosion-resistant metallic reinforcing bar, the superior reinforcing material, for at least the next decade."[80]

Ganz[31] states that: "…It has been proposed to replace prestressing steels with non-corrosive fiber reinforced plastic materials. It is the author's [Ganz] opinion that these materials still have to go through a long development until they can provide performance and reliability for post-tensioning applications comparable with well protected prestressing steel.

Gaubinger[81] states that: "…a common use [of CFRP tendons] is presently prevented by two main facts: high costs of manufacturing and the development of a suitable anchorage.

2.4.9 Anchorage Protection

As described by West[33], "the post-tensioning anchorages and end stubs of the strands must be carefully protected. Although anchorage corrosion may lead to failure of the anchorage, bond between the tendon and concrete will prevent a complete loss of prestress. Multistrand anchorage systems may be fitted with a sealed end cap to protect strand ends. The cap is grouted or filled with corrosion inhibiting grease. Not all multistrand post-tensioning systems include an end cap. Anchorages are commonly recessed in a pocket at the end or edge of the concrete element. Corrosion protection for the anchorage normally consists of filling the anchorage recess or pocket with mortar or concrete. Common practice is to coat the anchorage and pocket surfaces with an epoxy bonding agent prior to filling the anchorage pocket with a non-shrink mortar.

The location of the anchorage within the structure can also play a role in corrosion protection and corrosion damage. In many structures, the anchorages are located at the ends of structural elements below expansion joints, or at exterior member ends or slab edges. The location of post-tensioning anchorages is often dictated by the method of construction. In instances where the anchorage can not be located away from a possible source of aggressive agents, the anchorage must be detailed to provide multiple layers of corrosion protection. The Concrete Society (U.K.) Technical Report No. 47 on bonded post-tensioned bridges[22] provides suggestions for anchorage protection details. The report discusses two approaches for anchorage protection. The first is to provide an anchorage that is not encased in mortar or concrete after stressing. Exposed anchorage hardware is protected by end caps and waterproof membrane, and has the advantage that the anchorage can be readily inspected for corrosion damage. The second approach provides a higher level of corrosion protection at the expense of inspectability by recessing the anchorage in a filled pocket. Details of multilevel corrosion protection for this form of buried anchorage are shown in Figure 2.17. The details of the member end can also be designed to minimize contact with moisture and chlorides draining through expansion joints, as shown in Figure 2.18. The member end is detailed to prevent water from dripping onto the anchorage region. An abutment gallery is provided to allow inspectors to gain access to the anchorage."

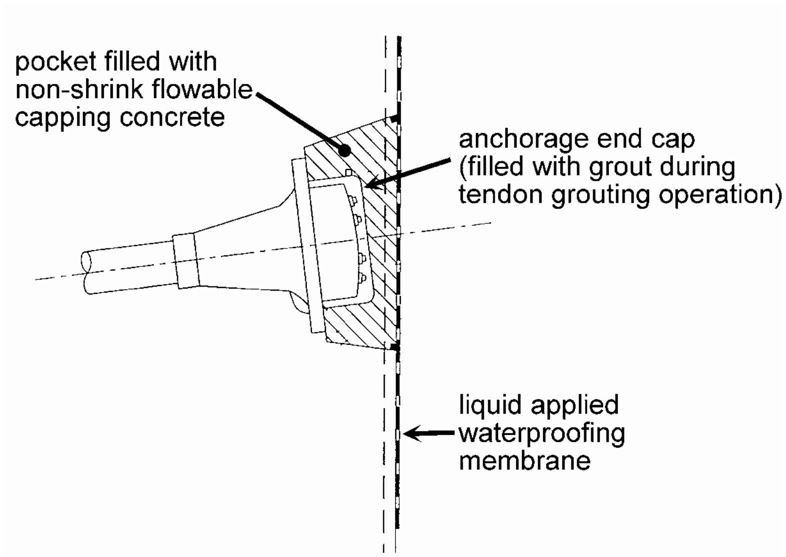


Figure 2.17 Multi-Layer Corrosion Protection for Buried Post-Tensioning Anchorages[22]

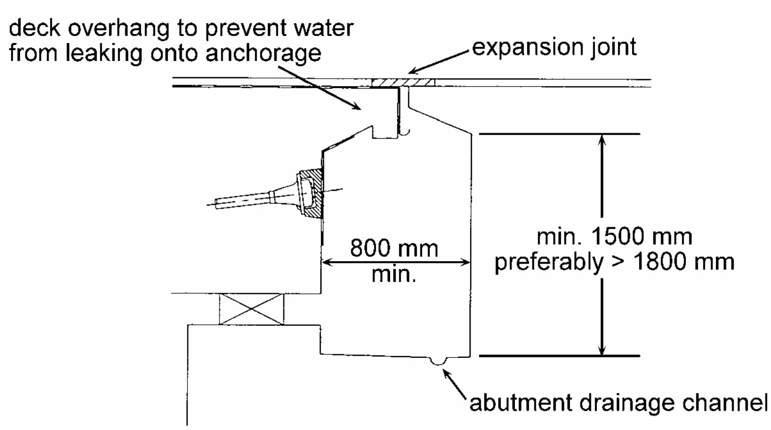


Figure 2.18 Member End Details for Anchorage Corrosion Protection[22]

2.4.10 Encapsulated and Electrically Isolated Systems

In a Electrically Isolated Tendon (EIT), the prestressing steel is entirely isolated not only by a robust plastic duct, but also at the anchorages. In this way, the EIT system is used to protect tendons from the effects of stray currents which could cause hydrogen embrittlement and pitting corrosion to the prestressing steel.[31,29]

Up to now, about 100 bridges have successfully been constructed using robust plastic ducts of which in over 20 bridges electrically isolated tendons have been installed.[29]

2.4.11 Precast Segmental Construction – Joint Detail

As stated by Moreton[7] "The U.K. "moratorium" is still in effect for "internal, grouted tendons with discontinuous (poorly sealed) ducts. This is broadly interpreted (or misinterpreted) to mean that only external tendons are allowed for precast segmental construction in the United Kingdom. This is not so. The concern of the Highways Agency is for the integrity of the ducts through the precast joints – that they should be completely sealed – and they require this via a performance specification."

Most countries consider epoxy resin to provide sufficient protection to post-tensioning tendons crossing joints between precast segments. However, the UK has specified full encapsulation for the tendons by plastic in these joints.[64]

In response to this problem, a new commercial coupler device is now available to connect plastic prestressing ducts used with match-cast joints in precast segmental construction.[76] The new coupler is suppose to yield good results in precast segmental construction. However, testing is still required. To this respect, Raiss[82] has stated that: "Pending further experience with this system, it may be prudent to assume that some percentage of the internal tendons is lost (say 5-10%)." However, this could be still unconservative until new research is done.

2.4.12 Temporary Corrosion Protection

Without further protection measures, grouting should be done within one to about 4 weeks in aggressive and benign climatic conditions, respectively. If grouting needs to be delayed beyond the above proposed intervals, particular methods need to be provided for post-tensioning tendons.[31]

While the use of Rust-Ban 310 is still under investigation (by EMPA – Swiss Federal Laboratory for Materials and Testing and ASTRA – Swiss Federal Highways Administration) it seems that it provides good temporary protection. The solution is applied at the factory of the strand supplier and sufficient time needs to be permitted to let the solution dry on the tendon.[31]

In case unprotected tendons were installed and grouting cannot be achieved within the accepted time frame, the tendons can be protected by blowing dry air through the duct.[31]

2.4.13 Inspection Practices

The American Segmental Bridge Institute (ASBI) has developed a three-day training program for the Certification of grouting technicians. This program has been developed to train the Inspectors in theory and field procedures to achieve a properly grouted structures.[21] The training manual contain updated and detail information on grouting materials, procedures and specifications.

2.4.14 Monitoring

Various techniques are being used to monitor the condition of existing post-tensioned bridges. However, they are not totally satisfactory, all of them having some limitations.[51]

Some non-destructive testing (NDT) and methods that could be used include:[51] georadar, potential mapping, impact-echo, remanent magnetism, radiography, reflectometry, ultrasonic, acoustic.

In addition, the use of electrically isolated tendons have proved successful.[51]

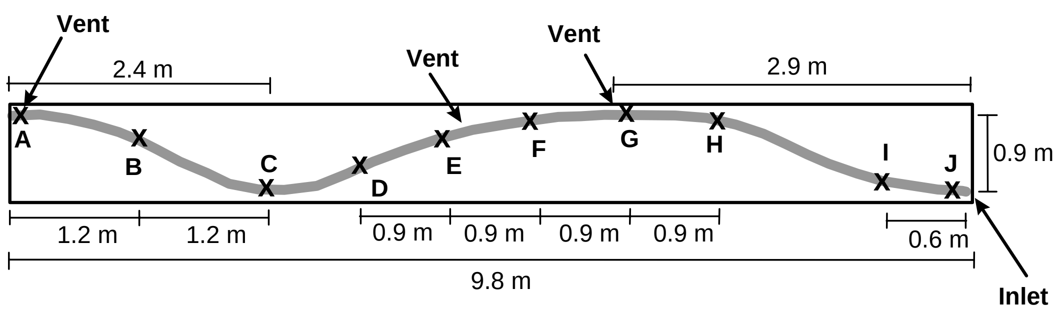
Chapter 3: Project Summary and Conclusions

Preliminary design guidelines were included in Report 0-1405-5 based on the first four reports from Project 0-1405. The findings of the literature review and experimental work were used to develop recommendations and implementation measures for post-tensioned bridge substructures. The durability design process was described, and guidance was provided for assessing the durability risk and for ensuring protection against freeze-thaw damage, sulfate attack and corrosion of steel reinforcement. The project continued with emphasis in the corrosion risk of post-tensioning systems and results are included herein.

In the following sections the final results and findings from improved and high-performance grout studies, macrocell corrosion tests, large scale beam corrosion tests, and large scale column corrosion tests, are included in detail.

3.1 Summary and Conclusions from Improved and High-Performance Grout Studies

The cement grout injected into the tendons in post-tensioned bridge structures has the important dual role of providing bond between the strands or bars and the concrete, as well as providing corrosion protection to the prestressing strands (or bars). An optimum grout combines good corrosion protection with desirable fresh properties so that the ducts can be completely filled with ordinary grouting techniques. Numerous grouts were tested in three phases of testing to develop a high performance grout for corrosion protection. The testing phases included fresh property tests, accelerated corrosion tests, and a large-scale clear draped parabolic duct tests (see Figure 3.1) that allowed observation of the grout under simulated field conditions. Observations and conclusions from each of the three test phase are given below along with the recommended grouts.



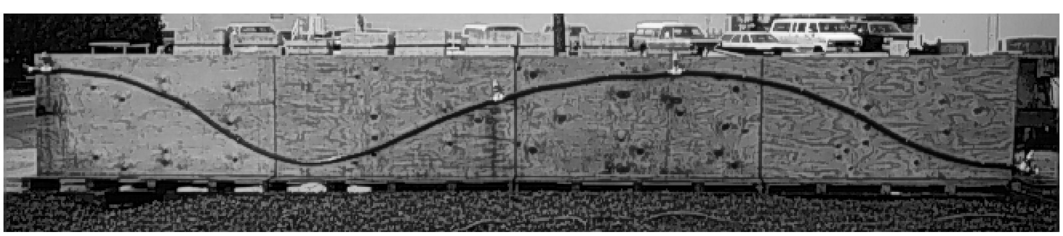


Figure 3.1 Large-Scale Clear Draped Parabolic Duct Test[34]

3.1.1 Fresh Property Tests

A wide range of trial grouts made with various water cement ratios and admixture combinations were subjected to a series of standard tests. These indicated:

- Increase with the addition of fly ash
- Decreased with the addition of silica fume

- Required the use of superplasticizer at low water-cement ratios
- Reduced with the addition of fly ash or silica fume
- Increased with the addition of superplasticizer
- Required an anti-bleed admixture to pass this test

3.1.2 Accelerated Corrosion Tests

An accelerated corrosion test method, ACTM, (See Figures 3.2 and 3.3) was used to compare a wide range of variables in order to determine the onset of corrosion. Typical results from the accelerated corrosion tests are shown in Figure 3.4.

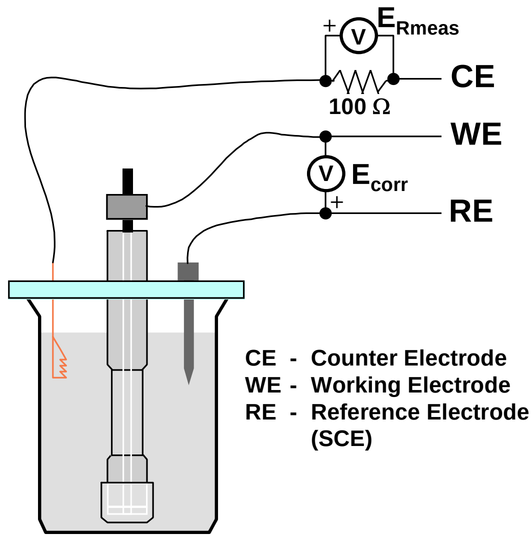


Figure 3.2 Schematic of ACTM Station[34]

Figure 3.3 ACTM Test Setup[34]

These ACTM tests indicated:

- Corrosion protection increases with lowered water-cement ratio
- Corrosion protection decreases with the addition of chemical admixtures including a calcium nitrite corrosion inhibitor
- A 30% fly ash grout with a 0.35 water-cement ratio had excellent performance (over 40% increase in average time to corrosion compared to a 0.40 water-cement ratio plain grout) and is recommended for use in most horizontal application.
- A 2% anti-bleed grout with a 0.33 water-cement ratio had good performance (average time to corrosion was similar to that of a 0.40 water-cement ratio plain grout) and is recommended for use in most high bleed vertical applications.

- The standard TxDOT grout had below average performance (average time to corrosion was lower than for a plain grout at the same water-cement ratio).
- Using prestressing strand from different spools can alter the test results. When comparing grout designs, the strand used should be consistent among all grouts.

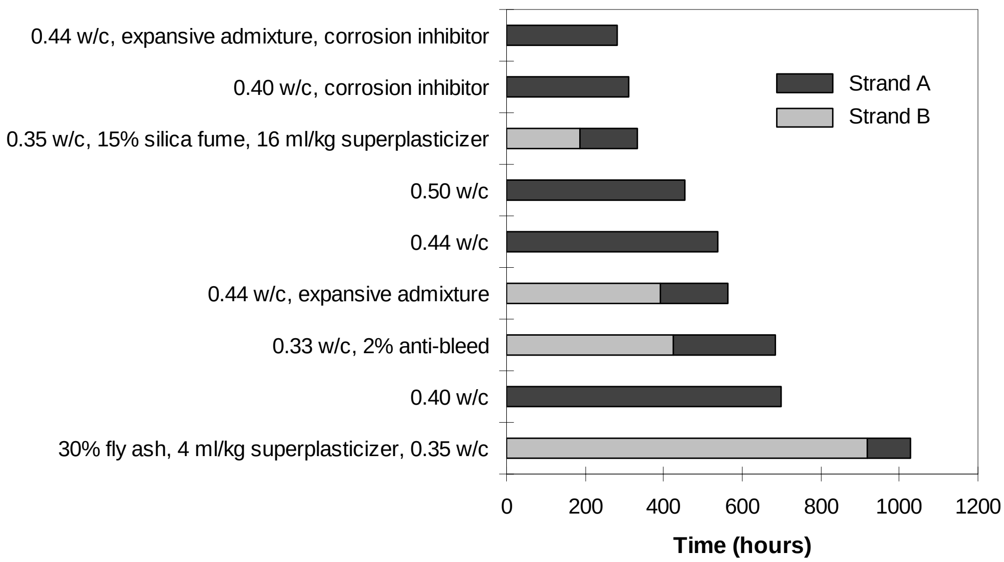


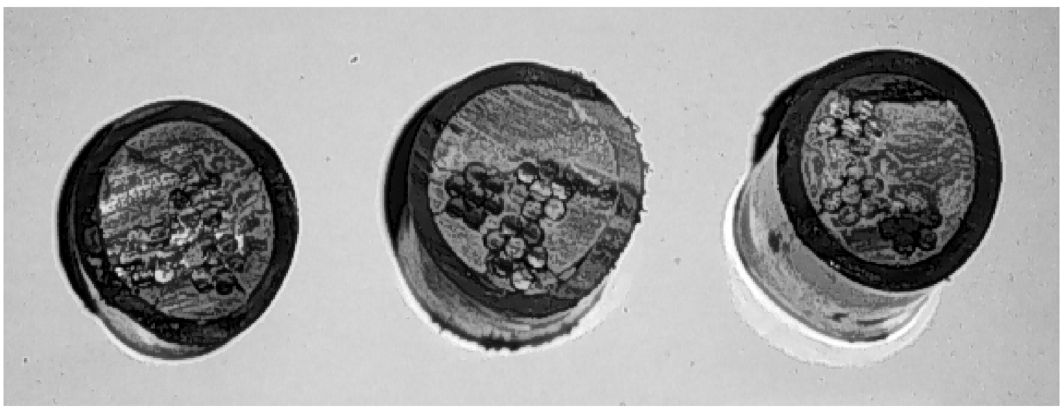
Figure 3.4 Comparison of ACTM Average Times to Corrosion[34]

3.1.3 Large-Scale Duct Tests

Observation of grouts during pumping and setting (See Figures 3.5 and 3.6) as well as examination of slices cut from the large scale grouted ducts (See Figure 3.7) indicated:

Figure 3.5 TxDOT Grout – Bubbles Traveling Toward Intermediate Crest[34]

Figure 3.6 Beginning of Void Formation at Intermediate Crest[34]



Anti-bleed Grout	*Fly Ash Grout*	*TxDOT Standard Grout*
(0.33 w/c, 2% Sikament)	*(0.35 w/c, 30% Fly Ash, 4 ml/kg Rheobuild)*	*(0.44 w/c, 0.9% Intraplast-N)*

Figure 3.7 Comparison of Slices from the Intermediate Crest[34]

3.2 SUMMARY AND CONCLUSIONS FROM MACROCELL CORROSION TESTS

Thirty eight macrocell specimens were used to investigate the corrosion protection of internal tendons at segmental joints (See Figures 3.8 and 3.9). Half of the specimens were autopsied after approximately four and a half years of highly aggressive exposure and preliminary conclusions were reported.[30] The variables analyzed during the testing program included: joint type (dry or epoxy), duct type (galvanized steel or plastic), grout type (3 grouts with differing additives) and level of joint compression (3 different levels). The second half of the specimens were autopsied with over eight years of very aggressive exposure. Example of macrocell condition prior to autopsy is shown in Figure 3.10. Numerous conclusions can be drawn.

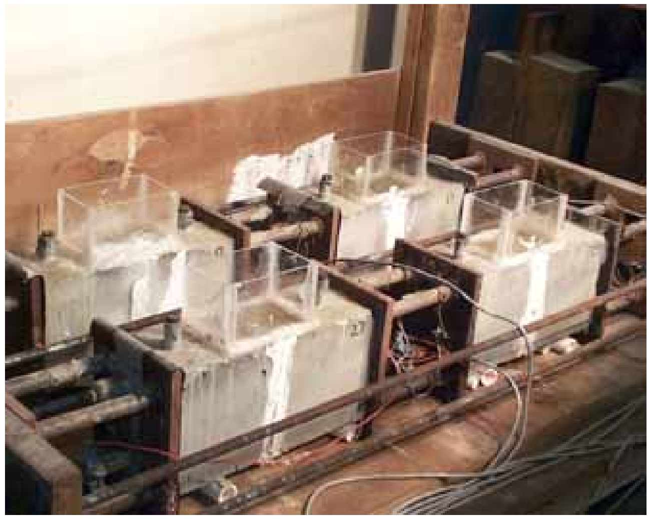


Figure 3.8 Macrocell Test Setup[83]

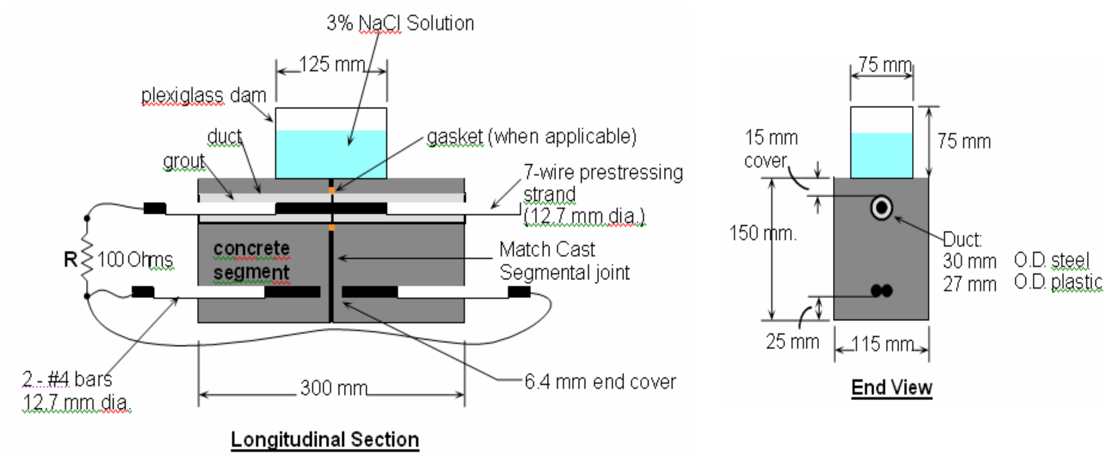


Figure 3.9 Macrocell specimen details[83]

Figure 3.10 Example of Specimen Condition Prior to Autopsy Specimen DJ-S-L-NG-2[83]

3.2.1 Overall Performance

- Superiority of plastic ducts was evident. Specimens with plastic duct had the best overall performance (quantified in terms of strand, mild steel and duct corrosion).
- All galvanized steel duct specimens showed some degree of duct corrosion, from moderate uniform corrosion up to severe duct destruction.
- Thin epoxy joints provided substantially improved corrosion protection when compared to dry joints. Incompletely filled epoxy joint performance was very similar to that of a dry joint.
- Post-tensioning strands were corroded in all specimens, from light uniform corrosion to moderate pitting.
- Mild steel bars were corroded in seventeen out of the nineteen specimens. One third of those had from moderate corrosion to severe pitting.
- In many instances, the epoxy coverage, provided on the strand and mild steel bars to limit the exposed length of the anode and cathode, failed to provide complete corrosion protection to these areas. Epoxy paint peeled off in many instances allowing for moisture and chloride ingress. Corrosion under the epoxy paint was in many cases comparable to the corrosion condition in the exposed lengths. Among other effects, this affected the current density calculations.
- Metal loss calculations based on current density calculations failed to indicate the amount of corrosion in the specimens.

3.2.2 Assessing Corrosion Activity Using Half-Cell Potential Measurements

Half-Cell Potentials were taken at two week intervals at the start of the wet period and at the start of the dry period. All measurements were performed according to ASTM C876[84] using a saturated calomel electrode (SCE). In all cases the prestressing tendon was not in contact with the galvanized duct, and it was considered that the segmental joint allowed for ion movement. However, while HC-Potentials in dry joint specimens had a good correlation with forensic examination results, they failed to detect corrosion activity in six out of nine epoxy joint specimens, and in one epoxy joint specimen with gasket.

With respect to testing variables, the following conclusions are drawn based on Half-Cell Potential Data:

- Epoxy joint specimens showed less probability of strand corrosion than dry joint specimens.
- Macrocell specimens with plastic ducts (discontinuous) at the joint showed less probability of strand corrosion than similar specimens with galvanized steel ducts.
- Dry joint specimen data indicated less probability of strand corrosion with increasing levels of joint precompression. This trend was not clearly shown in epoxy joint specimens.
- Dry Joint specimens with Corrosion Inhibitor (Calcium Nitrite) showed less probability of strand corrosion with respect to specimens with Normal Grout.

3.2.3 Segmental Joints

To address typical North American practice, dry joints and epoxy joints, with and without gaskets, were selected for investigation in this testing program. All joint types were match-cast. The AASHTO Guide Specifications for Segmental Bridges[85] does not permit the use of dry joints with internal tendons. However, dry joints were included as a worst case scenario for comparison purposes. The thin epoxy-jointed specimens were assembled according to the standard practice. In the epoxy/gasket joint, a foam gasket was glued to the face of one segment around the duct opening prior to application of the epoxy. Forensic examination after eight years of exposure included: seven specimens with dry joints, nine specimens with epoxy joints and three specimens with epoxy joints with gasket. The conclusions are as follows:

- All galvanized steel ducts and prestressing strands in the nineteen specimens showed some degree of corrosion. The higher corrosion ratings were obtained from dry joint specimens with galvanized steel ducts and normal grout. Ducts in these specimens were extremely corroded, with corrosion centered at the joint, and with concrete cracking in the top of the specimen. In general, dry joint specimens showed increased chloride penetration and increased corrosion of galvanized steel duct, prestressing strand and mild steel reinforcement. These results show that dry joints do not provide adequate corrosion protection for internal tendons in aggressive environments.
- Sound epoxy joint specimens with galvanized steel ducts showed moderate to very severe duct corrosion centered away from the joint. Clear cover for specimens was small (five eights to three quarters of an inch), significantly lower than would be allowed by specifications. However, the test results indicate the potential corrosion problems when using galvanized ducts in aggressive environments if chlorides penetrate the concrete cover away from the epoxy joint.
- Thin epoxy joints provided substantially improved corrosion protection when compared to dry joints. However, test results showed that poor epoxy filling at the joint is extremely detrimental to the performance of the duct, the prestressing strand and the mild steel reinforcement. Incomplete filled epoxy joint performance was very similar to that of a dry joint.
- Corrosion of mild steel in some epoxy joint specimens was found to be the result of an external source of moisture and chlorides rather than from penetration at the epoxy joint or through the concrete. This conclusion was reinforced with chloride levels measured at the joint and away from the joint. These findings reinforce the need to provide adequate clear cover over the ends of longitudinal bars in the segments, even if external post-tensioning is used.
- In some cases, the use of gaskets in epoxy jointed specimens prevented complete epoxy coverage of the joint. This condition could worsen under field conditions.

3.2.4 Ducts for Internal Post-Tensioning

Two duct types were investigated; standard galvanized steel duct and plastic duct. Due to size limitations, PVC pipe was used for the plastic duct. Test results indicated:

- Superiority of plastic ducts was evident. Strand encased in plastic ducts showed only light corrosion and discoloration. Specimens with plastic duct had the best overall performance (quantified in terms of strand, mild steel and duct corrosion).
- Galvanized steel duct was corroded in all specimens. Severe corrosion and large duct destruction was observed in dry joint specimens. Such corrosion was often centered on the dry joint. Epoxy joint specimens showed moderate to severe duct corrosion. The corrosion was often centered away from the joint.
- Concrete cover in specimens was lower than allowed by specifications. However, test results indicate that these are potential corrosion problems when using galvanized steel ducts in aggressive exposures if chlorides penetrate the cover. Plastic ducts performed well in spite of the small cover.

3.2.5 Joint Precompression

Due to the small specimen size, the strand could not be post-tensioned effectively. To simulate precompression across the joint due to post-tensioning, the pairs of match-cast segments were stressed together using external loading frames. Three levels of precompression were selected: 5 psi, 50 psi and $3\sqrt{f'c}$ psi. The lowest level of 5 psi could represent the level of precompression encountered in a precast segmental column under self weight. The precompression of 50 psi is based on AASHTO Guide Specifications.[85] The highest precompression value corresponded to 190 psi for this testing program.

Eight out of the nineteen specimens (at eight years of exposure) had low precompression, seven medium precompression and four high precompression. Conclusions are as follows:

- Test results did not show a clear trend with respect to joint precompression when analyzing time to corrosion initiation and rate of corrosion in prestressing strands and mild steel bars. An isolated result for dry joint specimens with galvanized steel ducts and normal grout showed that at very high levels of precompression, there is an increased level of strand and mild steel protection. This result is not clearly shown for epoxy joint specimens with and without gasket.
- Galvanized steel duct corrosion in dry joint specimens also showed better performance with a higher level of precompression. However again, this result is not clearly shown in epoxy joint specimens. Precompression level is much important in dry joint specimens.

3.2.6 Grouts for Bonded Post-Tensioning

Three cement grout types were selected for evaluation; normal grout (plain cement grout, no admixtures, w/c = 0.40), grout with silica fume (13% cement replacement by weight, w/c = 0.32, superplasticizer added) and grout with a commercial calcium nitrite corrosion inhibitor (w/c = 0.40). The dosage of corrosion inhibitor used in this testing program was the same dosage normally used for concrete (approx. 20 liters/m^3 concrete). The Calcium Nitrite dosage was not adjusted to account for the higher cement content in grout. The testing program for the nineteen remaining specimens at eight years of exposure included thirteen specimens with normal grout, five with corrosion inhibitor and one with silica fume. Conclusions are as follows:

- Grout voids, due to entrapped air, bleed water, incomplete grout filling or lack of grout fluidity were detrimental not only to the prestressing strand, but also to the galvanized steel duct.
- Dry joint Specimens with Corrosion Inhibitor (Calcium Nitrite) added to the grout showed a lower strand corrosion rating (less strand corrosion severity) at eight years of exposure, than specimens with normal grout (in the order of seven times smaller). This trend was not clearly shown in epoxy joint specimens. This result contradicts those obtained at four and a half years of exposure where the most severe corrosion of the prestressing tendon was found where calcium nitrite corrosion inhibitor was used.
- Epoxy joint specimens with silica fume, corrosion inhibitor and normal grout had very similar performances. No clear distinction was evident.

3.3 SUMMARY AND CONCLUSIONS FROM BEAM CORROSION TESTS

Twelve out of the twenty-seven large scale beam specimens were fully autopsied to evaluate the effect of post-tensioning on durability and to evaluate the relative performance of a large number of corrosion protection variables. Two additional specimens were partially autopsied. Full autopsies for the remaining specimens will be performed at a future date. Specimen setup details are shown in Figures 3.11 and 3.12. Beams were fabricated in two phases in order to begin exposure testing on a portion of the specimens while the remaining specimens were being fabricated. In Phase I (16 beams), which started exposure testing in December 1997, researchers investigated the effect of prestress level and crack width and also included one of the high performance grout specimens. In Phase II (11 beams), which started exposure testing in December 1998, researchers investigated duct splices, grout type, concrete type, strand type, duct type, and end anchorage protection. An example of specimen condition prior to autopsy is shown in Figure 3.13. After the first full autopsy performed at four and a half years for six Phase I beams, and three and a half years for six Phase II beams, and partial autopsies performed to two Phase I beams, preliminary conclusions were drawn.

Figure 3.11 Beam Test Setup at North End of Ferguson Laboratory[37]

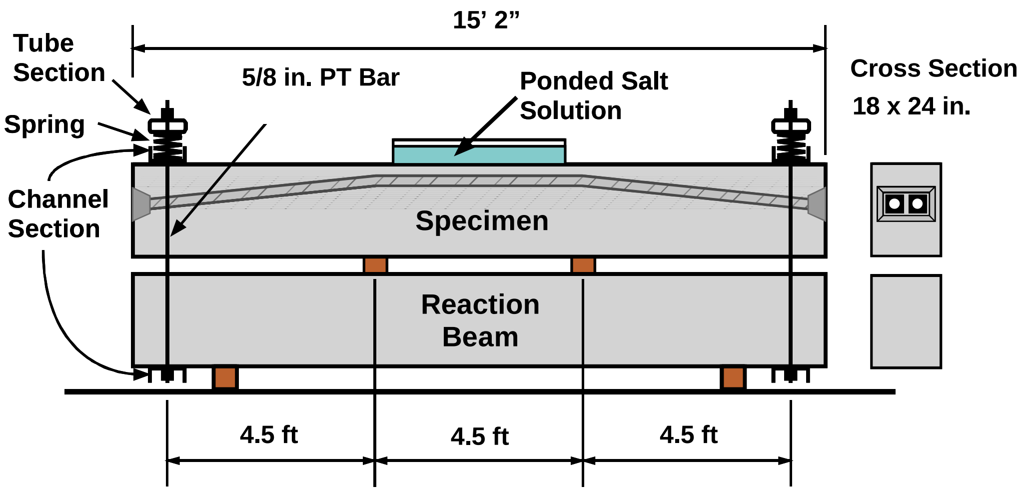


Figure 3.12 Test Setup[33]

Lateral (South) View *Top View (from South Side)*

Figure 3.13 Example of Specimen Condition Prior to Autopsy
Specimen 2.3 - Mixed Reinforced Beam under Constant Service Load[37,38]

3.3.1 Overall Performance

The variables selected for evaluation in this beam testing program fall into four main categories: level of prestress and crack width, concrete type, prestressing strand coatings and post-tensioning hardware protection. In addition, different post-tensioning duct splices were also evaluated. After the initial autopsies of the fourteen beams, the use of large scale beam specimens was found to be a very good method for determining the effect of most of these variables. Prestressing strand coatings and post-tensioned hardware protection will be evaluated at a future date, since they are included in the remaining specimens under exposure testing. Based on the autopsies performed to date, the following conclusions are drawn:

- Galvanized duct performed poorly. No plastic duct was used in the specimens of the first set of full autopsies.
- Bleed water voids were present in the ducts even after "good grouting procedures." Anti-bleed grout was not evaluated in the first set of full autopsies, but it is included in one of the remaining specimens for future autopsy.
- Voids from bleed water in grout were shown to be very detrimental to the duct.
- A clear trend was found with respect to cracking and mild steel corrosion. As cracking increased, stirrup and rebar corrosion increased. This trend was not clearly shown on strands, since strand ratings were all very low and close in value.
- Mixed reinforcing (2/3 PS) beams showed the worst corrosion resistance. The best performance was obtained from 100%S PS specimens, followed by 100%U PS specimens.
- Phase I beam results showed that there was a reduced risk of corrosion damage with increasing levels of prestress.
- High performance concrete specimens (low permeability, w/c=0.29) appear to perform better than class C fly ash concrete specimens. However, both appear to be effective in minimizing the chloride penetration through concrete.
- Industry standard duct splices as well as heat shrink duct splices do not seem to provide adequate corrosion protection.
- Duct splice damage did not show a direct correlation with the severity of corrosion.
- No difference was found between normal and fly ash grout. Low strand corrosion ratings on all specimens after autopsy, did not allow clear identification of the effect of different types of grout.

3.3.2 Load/Prestress Level versus Corrosion: The Effect of Cracking

The effect of cracking (width and number) on corrosion protection was an area of great emphasis in this experimental program. The effect of cracking was primarily investigated using standard variables and the sections that would be expected to crack under service loads. The range of crack widths investigated in this program were based on a survey of relevant literature performed by West[33] regarding critical crack widths for corrosion and recommended allowable crack widths. Consideration was also given to the applied moment-crack width behavior computed for the sections. Three different load levels were used: unloaded, service load, and temporary overloaded. The following conclusions are drawn:

- The specimen corrosion protection decreases as the applied load increases.
- Corrosion protection decreases with increasing cracking.
- An increase in transverse crack width produces a decrease in corrosion protection.
- Longitudinal or splitting cracks in the concrete surface are a clear indication of very severe corrosion within the member.

- The chloride content in the concrete is significantly higher at crack locations, and increases as the crack width increases.
- The specimen corrosion protection increases as the level of prestress increases.
- Mixed reinforcement (2/3 PS) beams showed the worst corrosion performance. Increasing the post-tensioning level from 2/3 PS to 100% PS significantly increased the corrosion protection.
- The corrosion protection of the 2/3 PS beam was much more similar to Non-PS beams, as opposed to 100% PS Beams.
- There was not a clear difference in the corrosion resistance among the fully prestressed beams designed with the ultimate strength method as compared to those designed with allowable stress method.

3.3.3 Fly Ash in Concrete

Concrete plays an important role in corrosion protection of steel reinforcement. One of the objectives of this research program is to evaluate the effectiveness of high performance concrete as a function of cracking. Three different concrete mixes were selected for comparison. The reference mix was the standard concrete: TxDOT Class C concrete. The alternates were a TxDOT Class C concrete with 25% Class F fly ash and a high performance concrete (0.29 w/c, 25% fly ash + superplasticizer). The following conclusions are drawn:

- Both the high performance concrete and the fly ash concrete beams showed good corrosion protection by minimizing the chloride penetration through the concrete.
- The high performance concrete tends to show a slightly better corrosion protection than the fly ash concrete, but the difference is not significant.
- No conclusions can be drawn on corrosion protection of the high performance concrete and the fly ash concrete with respect to the standard TxDOT concrete due to the unfortunate lack of directly comparable specimens at the time of the first autopsy.

3.3.4 Duct Splices for Galvanized Steel Duct

In most practical applications, the post-tensioning ducts must be spliced at some location. It was decided to compare industry standard (IS) splices to heat shrink (HS) splices and unspliced duct. The effect of damaged splices was also examined. The IS splice consisted of a 1 ft length of oversized duct placed over the contact butt splice of the ducts. Concrete is prevented from entering the splice by wrapping the ends with duct tape. The heat shrink splice consists of an 8-inch length of heat shrink tubing placed over the contact butt splice of the ducts. The original diameter of the heat shrink tubing was 4 inches. No mechanical connection was made between the two ducts being connected. The conclusions are as follows:

- The industry standard splice allowed moisture and chlorides to enter through the sides of the splice and get trapped between the duct and the splice due to inefficiency of duct tape.
- The heat-shrink splice also allowed moisture to enter through the sides and get trapped due to insufficient adhesion between the splice and the duct. It also traps bleed water from the grout.
- Damage inflicted on the duct splices did not show a direct correlation with the severity of corrosion.
- Neither the industry standard splice nor the heat-shrink splice appears to be a satisfactory duct splice for the corrosion protection of a galvanized steel duct.

3.3.5 High Performance Fly Ash Grouts

Two high performance grouts (a fly ash grout and an antibleed grout) were selected for investigation, in comparison with TxDOT standard grout. The fly ash grout specimen was autopsied, and results are reported herein. The antibleed grout specimen will be autopsied at a future date. Antibleed grout had a water-cement ratio of 0.33 with 2% cement weight of antibleed admixture. Based on the information to date, the following conclusions are drawn:

- The fly ash grout aided in the corrosion protection of the galvanized steel ducts
- The fly ash grout, in comparison to TxDOT standard grout, did not show an increase in corrosion protection of the prestressing strand. This result may be due to the strand ratings being very low and close in value. Several more years of exposure testing may be require to yield more conclusive results

3.3.6 Exposure Testing Results

Half-cell potential readings were measured using a saturated calomel reference electrode at the end of each wet cycle (once every four weeks). All measurements were performed according to ASTM C876.[84] In general, half-cell potential readings are inadequate in determining the severity of corrosion activity, but prove to be successful for relative comparison of specimens. The conclusions are as follows:

- There is an exact correlation in specimen performance between the greatest negative potential at the end of testing for autopsy beams and the time to corrosion
- Both half cell potential readings and corrosion rating graphs show the loaded Non-PS and 2/3 PS beams were the most corroded.
- Half-cell potential readings did not show a distinct correlation in high performance and fly ash concrete specimens with the corresponding corrosion ratings.

Corrosion rate measurements were taken four times during the exposure duration. Two types of equipment were used in this experimental program: the Pr Monitor and the 3LP. Measurements of the Phase I beams were taken after seven, twelve, fifteen and forty-seven months of exposure. Measurements of the Phase II beams were taken after 37 months of exposure. A final attempt to take corrosion rate measurements of all beams was made immediately prior to the forensic examination. This attempt was unsuccessful due to complications with the 3LP equipment. Corrosion rate readings did not show good correlation with forensic examination results. The presence of zinc in the galvanized steel ducts may have played a role in the erroneous results.

Chloride content was found to be a useful method in determining the onset of corrosion. However, there was not a direct relationship between the acid soluble chloride content at the bar/duct level and the severity of corrosion at time of autopsy.

3.4 SUMMARY AND CONCLUSIONS FROM COLUMN CORROSION TESTS

Five non-prestressed and five post-tensioned columns specimens were used to investigate corrosion mechanisms and chloride ion transport (“wicking effect”) in various column connection configurations and to evaluate column corrosion protection measures (See Figures 3.14 and 3.15). Variables included column to foundation connection (no dowel joint, doweled joint and post-tensioned joint), loading (no loading and service load – with combined moment and axial load), concrete type (TxDOT Class C concrete, and Class C Fly Ash concrete – 35% replacement by volume), prestressing bar coatings (uncoated, galvanized PT-bars, and epoxy coated PT-bars), and post-tensioning ducts (plastic and galvanized steel). Trickle water was used on one face of each column to determine the effect of salt water spray or dripping. Test specimen exposure started in July of 1996 and ended in January of 2003, after six and a half years. Full autopsies were performed at the end of testing (See Figure 3.16), and conclusions are as follows.

Figure 3.14 Column Test Setup [37]

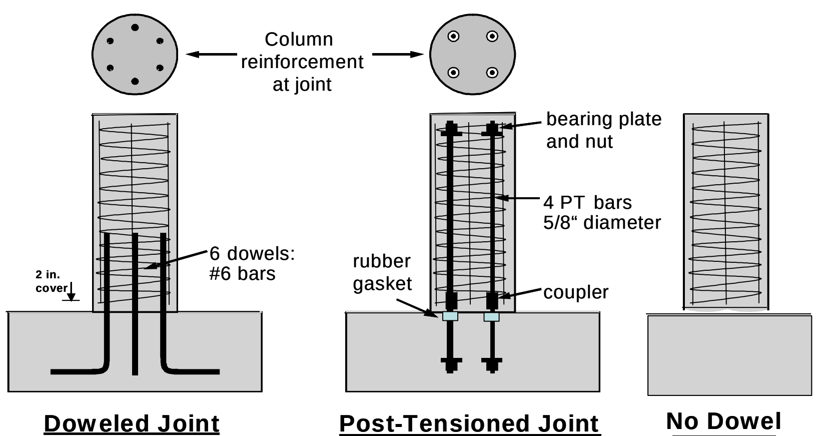


Figure 3.15 Column-Foundation Joint Configurations [37]

Figure 3.16 Column concrete Removal and Reinforcement Dismantling[37]

3.4.1 Post-Tensioning to Improve Corrosion Protection

The possible weak link in corrosion protection at the column-foundation interface was studied with three different configurations: no dowel joint, doweled joint and post-tensioning joint. Determination of the effect of post-tensioning on durability through precompression of the concrete and precompression of construction joints was one of the main objectives of this research series. The conclusions are as follows:

- Post-tensioned specimens did not show any distinct improvement in specimen performance at the column foundation interface, when compared to doweled specimens.
- Post-tensioning dramatically reduced the corrosion found on spiral reinforcement in the first 18 inches from the column base.
- Post-tensioned specimens under loading showed an increase in spiral and mild steel reinforcement corrosion protection when compared to non-loaded specimens.
- Post-tensioning reduced the risk of spiral corrosion due to saltwater dripping.
- Post-tensioning provided better resistance to the wicking effect, when acid-soluble chloride contents were compared to doweled and no-dowel specimens.

3.4.2 *Fly Ash as Partial Cement Replacement in Concrete*

TxDOT standard concrete mix Class C concrete was used in eight specimens (w/c = 0.45, type I/II Cement, f′c = 3600). In two columns, 35% of cement by volume (31% by weight) was replaced with Class C Fly Ash, with no other significant changes to the concrete mix. After autopsy, the following conclusions are drawn:

- Fly ash concrete did seem to provide enhanced corrosion protection to galvanized steel ducts.
- Spiral and mild steel reinforcement corrosion in non-prestressed specimens showed a better performance in Fly Ash concrete than in Class C concrete specimens. This trend was not clearly shown in post-tensioned specimens.
- Post-tensioned bar corrosion did not show any distinct trend with respect to the type of concrete in the specimen.

3.4.3 *Plastic Ducts for Post-Tensioning*

Standard galvanized steel ducts were compared to impermeable plastic ducts in three post-tensioned specimens: Class C concrete (unloaded and service load) and Fly Ash concrete under service load. In all cases uncoated post-tensioning bars were used. The conclusions are as follows:

- Although results are very limited, advanced galvanized steel duct corrosion at the column base, inside the rubber gasket, show the superiority of using plastic ducts.
- Corrosion in post-tensioned bars in plastic ducts and galvanized ducts was always at the column-foundation interface, where the plastic or galvanized duct was interrupted. Therefore, conclusions regarding duct performance based on post-tensioning bar corrosion are not possible. The ducts need an effective splice seal at all joints.

3.4.4 *Post-Tensioning Bar Coatings*

Two prestressing bar coatings were investigated: epoxy coated (according to ASTM A775-97) and zinc galvanized prestressing bars. The coated bars were compared directly to uncoated bars within individual specimens. In both cases, anchorage hardware was either epoxy coated or galvanized. The following conclusions are drawn:

- Epoxy coating or galvanized post-tensioning bars showed enhanced corrosion protection, with respect to plain post-tensioning bars.
- Coatings were not sufficient to stop corrosion from occurring at the column-foundation interface. Corrosion was very localized.
- Superiority of coated bars should not be concluded, since localized corrosion may accelerate deterioration at the local level, which in turn may result in unexpected failure.

3.5 MAJOR OVERALL CONCLUSIONS

Based on the results from the improved grout, macrocell, beam and column corrosion test series, the following major overall conclusions are drawn:

Improved and High-Performance Grouts

- A 30% fly ash grout with a 0.35 water-cement ratio had excellent performance and is recommended for use in most horizontal applications.
- A 2% anti-bleed grout with a 0.33 water-cement ratio had good performance and is recommended for use in most high bleed vertical applications.
- The standard TxDOT grout had below average performance.

Ducts for Internal Post-Tensioning

- The use of galvanized steel ducts appears unwise. Severe duct destruction and pitting was found in most specimens with this type of duct, as seen in Figure 3.17. From macrocell and column corrosion test results the superiority of plastic ducts was evident. The use of encapsulated anchorage protection systems appear very promising but cannot be conclusively evaluated until after final autopsies of the remaining beam specimens. However, in view of the substantial corrosion found with galvanized ducts, it seems like a prudent step to use plastic duct in any application that has even a moderate risk of exposure to chlorides.

Figure 3.17 Large Galvanized Steel Duct Destruction Due to Corrosion[37,38]

- Unspliced plastic ducts, such as those used in the macrocell and column series, showed better protection of the strands or bars when compared to unspliced galvanized steel ducts. The use of completely filled epoxy joints with unspliced plastic ducts showed very good protection. However, in field applications and highly aggressive environments it might be prudent to consider positive coupling of internal ducts at segmental joints if epoxy is not to be thoroughly distributed around the duct openings. The need for better splicing systems to avoid any moisture and chloride penetration and the corresponding localized strand or bar corrosion was particularly evident at construction joints.

Cracking and Joints

- Transverse cracking due to loading had a definite effect on corrosion damage. As cracking increased, reinforcement corrosion increased. Larger crack widths and crack density were found to be the cause of very severe localized and uniform reinforcement corrosion activity. (See Figure 3.18)
- Longitudinal or splitting cracks always indicated very severe corrosion within the member, as shown in Figure 3.19.
- Sound epoxy joint filling is mandatory to prevent moisture and chloride ingress. Dry joints and incompletely filled epoxy joints in the macrocell specimens showed very poor performance. (See Figure 3.20) Similar results were observed in the column tests at the column bases for the non-doweled specimens. Dry joints performed as preset cracks.

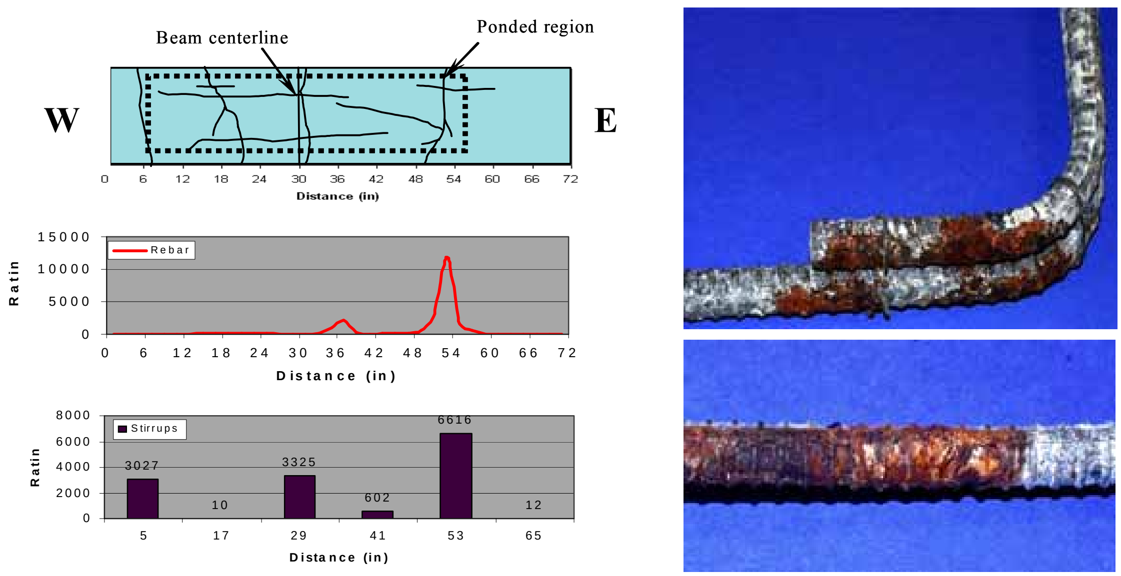


Figure 3.18 Effect of Transverse Concrete Cracking Due to Service Loading on Reinforcement Corrosion[37,38]

Figure 3.19 Example of Longitudinal (Splitting) Crack in Macrocell Specimen[83]

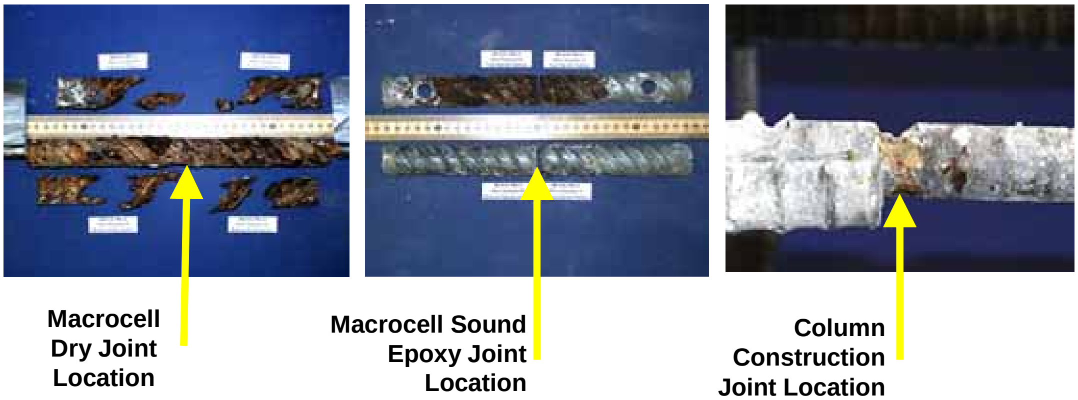


Figure 3.20 Segmental or Construction Joint Performance[37]

Levels of Post-Tensioning

- As the level of post-tensioning or concrete precompression increased, the corrosion protection increased. Mixed reinforced beams showed substantially more corrosion than fully prestressed members, as shown in Figure 3.21. Crack control and concrete precompression are definite factors in reducing corrosion risk. Lower permeability due to increased precompression also provided better resistance to wicking effects.

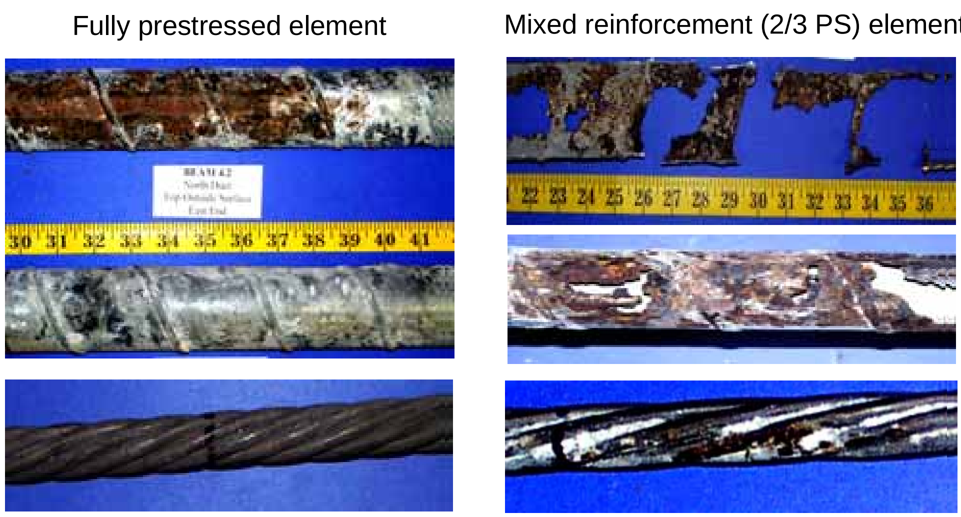


Figure 3.21 Corrosion in Galvanized Steel Duct in Beams with Different Levels of Post-Tensioning[37,38]

Concrete Type

- High performance concrete appears to be effective in minimizing the chloride penetration through concrete.

Concrete Cover

- Small concrete cover was clearly shown to be detrimental to reinforcement performance. When segmental joints allowed for moisture and chloride ingress, the smaller cover typical at the joint faces increased the corrosion activity in the reinforcement. Similar results were obtained at the base of the column specimens when analyzing spiral performance.

<u>Galvanized Ducts Splices</u>

- Neither the industry standard splice nor the heat-shrink splice appear to be a satisfactory duct splice to prevent moisture and chloride ingress to the grout for the corrosion protection of a galvanized steel duct. (See Figure 3.22.) Development of a much more effective duct splicing technique should be a high priority.

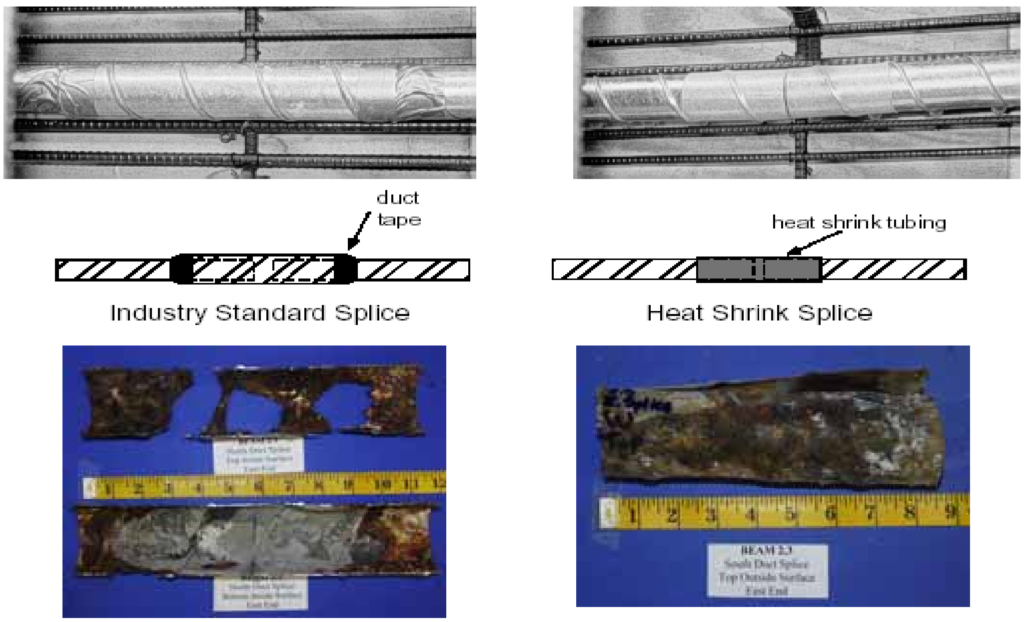
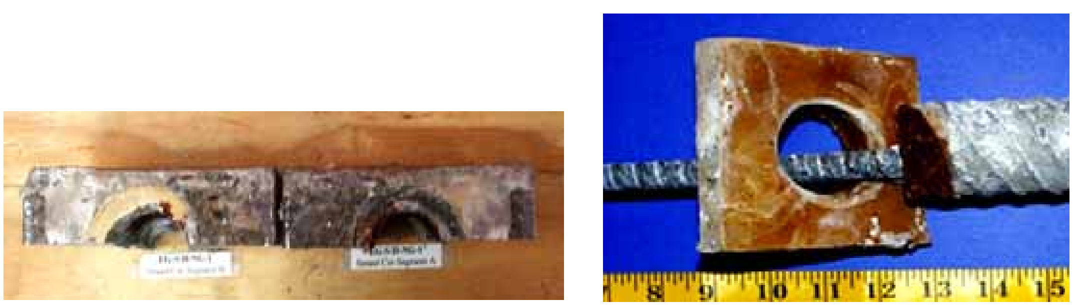


Figure 3.22 Duct Splice Performance[37]

<u>Gaskets for Post-Tensioning</u>

- The use of supplementary gaskets in the joints to avoid epoxy filling of the ducts, in the case of the macrocell specimens; or the use of rubber gaskets to seal the duct ends, in the case of the column specimens, were detrimental to the performance of the specimens. Both gaskets allowed for moisture and chloride ingress. (See Figure 3.23.)
- Gaskets used at the duct ends trapped moisture and produce crevice corrosion in the galvanized steel ducts.



Incomplete epoxy coverage in Epoxy/Gasket Joint

Rubber Gasket at the end of Galvanized Duct

Figure 3.23 Effect of the Use of Rubber Gaskets[37]

Grouts for Post-Tensioning

- The standard TxDOT Class C grout performed poorly in all specimens under evaluation. Better results were obtained with corrosion inhibitor added to the grout, and the use of fly ash. However, grout voids were not avoided with these grout mixes. The use of antibleed grouts appear promising but result cannot be conclusive until final autopsies of the remaining beam specimens.
- Grout voids, due to entrapped air, bleed water, incomplete grout filling or lack of grout fluidity showed to be detrimental not only to the prestressing strand, but also to the galvanized duct. In addition, bleed water was found to be detrimental to the galvanized ducts when using heat-shrink or industry standard splices. (See Figure 3.24.) Antibleed and better thixotropic grouts should be considered.
- Stringent grouting procedures must be enforced.
- Grout is not prestressed and often cracks where found under service loading. Figure 3.25 shows the effect of grout cracking.

Figure 3.24 Bleed Water Voids and Duct Corrosion[37]

Figure 3.25 Effect of Grout Cracking[37]

Post-Tensioning Bars or Strands

- Post-tensioning bar coatings (epoxy or galvanized) showed enhanced general corrosion protection, with respect to plain post-tensioning bars. However, under very severe localized attack, as in a crack or joint location, corrosion activity is severe, which in turn may result in unexpected failure. (See Figure 3.26)

- The use of epoxy-coated and galvanized strands will be more conclusive after final evaluation of the remaining beam specimens.

Uncoated Pt Bar *Epoxy Coated Pt Bar* *Galvanized Pt Bar*

Figure 3.26 Bar Coating Performance when Subjected to Localized Attack[37]

<u>Exposure Testing</u>

- Of the exposure testing methods used: half-cell Potential readings, chloride content determinations and corrosion current readings, only the first two showed some degree of correlation with forensic examination results.

3.6 Recommended Changes and Procedures

The following recommendations are given for consideration in similar experimental programs:

- When using the modified ASTM G-109 macrocell specimens, a few standard ASTM G-109 specimens with mild steel reinforcement only should be constructed for comparison.
- Where possible, when analyzing several variables in similar experimental programs, at least three specimens should be considered for each variable to clearly validate the results.
- Connection wires used for non-destructive measurements should be protected against the outdoor environment to avoid possible deterioration and corrosion that would increase resistivity. Also, wiring systems should be protected against salt water contact while filling up the Plexiglas containers for the aggressive exposure of the corrosion specimens.

3.7 Directions for Future Research

After the research results had been summarized, it is concluded that post-tensioning in concrete structures can provide enhanced durability, besides the well known structural and economical benefits. However, to ensure that the post-tensioning system is well protected against corrosion or deterioration, further research is needed in some specific areas. The authors recommend additional testing in the following fields:

- Use of encapsulated and electrically isolated systems
- Use of improved grouts for post-tensioning and better grouting procedures
- Use of post-tensioning duct couplers
- Development of better splice systems for galvanized ducts that might be used in non-aggressive environments.
- Use of impermeable surface membranes
- Development of better non-destructive methods for determining corrosion activity within post-tensioned concrete members.
- Use of improved concrete mix designs
- Use of improved strand or bar materials

Durability design guidelines for durable bonded post-tensioned concrete tendons are the most important implementation directed aspect of the research program. Results are based on specimens that were purposely placed under a very aggressive environment. By applying salt water in continuing two week wet and dry cycles, a condition simulating a harsh and extended service application was introduced. It cannot be directly related to any specific real life exposure age.

Interim design guidelines to reduce risk from various forms of attack, including corrosion risk, were developed and published as explained in Section 1.3.4., based on research results up to 1999. After full forensic examinations of all remaining specimens in the macrocell corrosion test series, all specimens in the large scale column corrosion tests series and approximately half of the specimens from the large scale beam corrosion test series, the following items are recommended for immediate implementation, with emphasis in reducing corrosion risk.

CHAPTER 4: IMPLEMENTATION RECOMMENDATIONS

4.1 MEASURES FOR IMPLEMENTATION FROM IMPROVED AND HIGH-PERFORMANCE GROUT STUDIES

Grout studies generated the following findings for immediate implementation:

Post-Tensioned Tendons with Small Rises

For post-tensioned tendons with a rise less than 5 meters, the present TxDOT standard grout should be replaced by:

- **0.35 water-cement ratio, 30% cement weight replacement fly ash (class C), and 4 ml/kg superplasticizer (similar and equal to Rheobuild 1000)**

This grout is recommended for situations requiring a high resistance to corrosion without extreme bleed conditions (vertical rise of less than 1 meter). This grout may also be appropriate for larger vertical rises (1-5 m) but field testing should be performed on a case by case basis.

Post-Tensioned Tendons with Large Rises

For post-tensioned tendons with large rises (5-38 m), the present TxDOT standar grout should be replaced by:

- **0.33 water-cement ratio, 2% anti-bleed admixture (similar and equal to Sikament 300 SC)**

This grout is recommended for situations requiring a high resistance to bleed (vertical rises up to 38 m) along with good corrosion protection. The maximum vertical rise recommended was based on results of the Gelman pressure test. The grout was not actually tested in a 38 m vertical rise.

4.2 MEASURES FOR IMPLEMENTATION FROM MACROCELL CORROSION TESTS

Macrocell test results generated the following findings for immediate implementation to improve corrosion protection for precast segmental construction.

Duct type

- Plastic ducts for post-tensioning should be used in all situations where even moderate aggressive exposure may occur.

Joint Type

- Epoxy joints should always be used with internal prestressing tendons.
- Dry joints should be avoided with external prestressing tendons in even moderate aggressive exposures, to protect segment mild steel reinforcement at joints and to block entry of chlorides that might be transported to locations of flaws in external tendon sheaths or anchors.
- Stringent inspection and construction practices must be exercised to guarantee good epoxy filling at the joints and complete grouting.
- Gaskets in epoxy joints should be avoided since there is a potential for incomplete epoxy coverage of the joint. Preferred practice with epoxy joints is to utilize a thorough swabbing of tendon ducts immediately after initial segment placement and stressing to seal the duct edges at the joint. Tightness of the joint should be checked by air pressure testing. Carefully coupled ducts are an alternative as long as a positive seal is obtained.

Grout type

- Calcium Nitrite Corrosion inhibitor added to the grout had little effect on the onset of corrosion but did seem to provide enhanced long-term strand corrosion protection.

4.3 Measures For Implementation From Beam Corrosion Tests

After final autopsies of twelve out of twenty-seven beam specimens and partial autopsies of two beam specimens, research results generated the following findings. Final autopsies of the remaining beam specimens will be more conclusive for strand, duct and grout types, and also for the use of encapsulated anchorage systems.

Level of Prestress

- Mixed reinforcement members should not be used in aggressive exposures unless special provisions are made to effectively seal cracks and concrete cover from exposure to chlorides.
- Fully prestressed members are recommended in aggressive environments to delay moisture and chloride ingress.
- Post-tensioning systems need additional protection above the current typical practice when in aggressive environments. In particular the use of galvanized duct appears unwise. The use of plastic ducts and encapsulated anchorage protection systems appear promising but while plastic duct was clearly superior in the macrocell specimens the use in the beam specimens cannot be conclusively evaluated until after final autopsies of the remaining beam specimens.

Concrete type

- High Performance Concrete (low permeability concrete, w/c = 0.29) is recommended in aggressive environments due to the significantly reduced permeability and crack control.
- Fly ash (Class C) concrete with a higher water cement ratio (w/c = 0.44) may also be considered when the environment is less aggressive.

Duct Splices for Galvanized Ducts

- Neither the standard industry practice of duct taped sleeves nor heat shrink splices should be considered as watertight
- Better systems than industry standard or heat-shrink splices for galvanized steel ducts should be investigated and developed if galvanized duct continues to be used in non-aggressive environments.

High Performance Fly Ash Grout

- Standard Class C grout with fly ash is not recommended.
- The use of antibleed admixture appears promising but cannot be conclusively evaluated until after final autopsies of the remaining beam specimens.

Grouting Procedure

- Stringent grouting procedures should be enforced during construction.

Plastic Chairs

- Fully plastic chairs are recommended for use throughout the substructure to eliminate corrosion damage. Chairs or bolster strips that contain any steel should be avoided.

4.4 MEASURES FOR IMPLEMENTATION FROM COLUMN CORROSION TESTS

After full autopsy of all ten column specimens, research results generated the following findings to be implemented for partially submerged columns or columns exposed to saltwater dripping:

Duct Types

- Plastic ducts should be used to better protect post-tensioning bars. However, better sealing materials or splices should be used or developed, to couple or seal the duct at construction joints or "dead" ends and protect the post-tensioning bar.

Substructure Prestressing

- Column elements should be prestressed, to improve spiral and rebar corrosion protection. However, designers should not rely entirely on post-tensioning to provide adequate corrosion protection at the cold joint. Other protection measures should be investigated.

Concrete Type

- None of the column specimens had high performance concrete, so no conclusions can be made about this material.
- Fly Ash concrete (w/c = 0.42) may be used to provide enhanced spiral, rebar and duct corrosion protection.

Gaskets

- Rubber gaskets are not effective to seal the duct "dead" ends and should not be used.

Post-tensioning Bar Coatings

- Galvanized steel bars or epoxy coated bars provide enhanced protection against uniform corrosion, but are susceptible to severe localized corrosion.

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