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CORROSION ACTIVITY IN PRECAST CONCRETE ELEMENTS AND CEMENTITIOUS CLOSURE POURS

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CORROSION ACTIVITY IN PRECAST CONCRETE ELEMENTS
AND CEMENTITIOUS CLOSURE POURS

A Thesis
Presented to
the Graduate School of
Clemson University

In Partial Fulfillment
of the Requirements for the Degree
Master of Science
Civil Engineering

by
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August 2013

Accepted by:
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ABSTRACT

The use of prefabricated elements in the construction of highway bridges has been a common practice in the United States since 1950s. Precast concrete adjacent box beam girder bridges are quite popular given that the precast elements not only have structural capacity to span across the supports, but also form the bridge deck. These bridges have generally performed well during the initial years after construction, but recent failures in Pennsylvania and Indiana have aroused a nationwide alarm to investigate the causes of failure and find solutions to repair these bridges or develop design for new replacement bridges. It was found that the stressed and non-stressed embedded reinforcements were severely corroded due to the ingress of chloride ions in the bridge deck through the surface cracks occurring within the closure pours (shear key) and at the interface of the joints between the precast elements and the shear key. The primary objective of this research is to find an easy and reliable technique to detect the initiation of corrosion in embedded rebar with minimal disturbance to the moving traffic and to investigate the condition of the rebar (active or passive with respect to corrosion) embedded in Ultra High Performance Fiber Reinforced Concrete (UHPFRC) so that precautionary measures and maintenance can be applied before the failure of more in-service bridges. The experimental investigation considered three different samples; ASTM modified G 109 samples (laboratory samples) which served as a control type specimen and large un-cracked and cracked specimens (field samples) which served as an actual prototype of a bridge deck. These samples were exposed to 3% of NaCl solution in the wet condition for duration of two weeks in alternate cycles of wet and dry period to accelerate the

corrosion process. Half-cell potential (HCP) tests and linear polarization resistance (LPR) tests were carried out on all the samples. HCP test was able to detect the accurate location of corrosion by measuring the corrosion potential in the rebar and LPR test was able to confirm the active or passive state of the rebar by measuring the corrosion current in the rebar. The results of both these tests were validated by observing the physical condition of the rebar which were obtained from cores extracted from each sample types. It was found that rebar were mainly corroded at the interface of the joints between two different materials and areas of the samples which were cracked from the bottom in case of cracked samples. The corrosion of rebar was most prominently observed in the UHPFRC with PVA fibers even at places without joints and cracks. More intensive investigation is needed which will require more time and applications of more detail electro-chemical techniques to arrive at a more confident conclusion.

DEDICATION

This thesis is dedicated to my parents, Yusufali and Sakina Udaipurwala and my sister Tasneem Udaipurwala and to my graduate advisors, Dr. Scott D. Schiff and Dr. Amir Poursaee. I could not have completed my research without their endless help, advice and support during my graduate course of time in Clemson University.

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TABLE OF CONTENTS

	Page
TITLE PAGE.....	i
ABSTRACT.....	ii
DEDICATION.....	iv
ACKNOWLEDGMENTS	v
LIST OF TABLES	ix
LIST OF FIGURES	xi
CHAPTER	
I. INTRODUCTION	1
1.1 Introduction.....	1
1.2 Corrosion of steel reinforcement bars.....	7
1.3 Objectives	10
1.4 Summary of approach	12
1.5 Outline of the thesis	13
II. BACKGROUND	15
2.1 Bridges	16
2.2 Corrosion and its mechanism.....	19
2.2.1 Kinetics and Thermodynamics of corrosion	22
2.2.2 Influence of cracks on corrosion.....	24
2.3 Corrosion measuring techniques.....	26
2.3.1 Half-cell potential test.....	26
2.3.2 Linear polarization resistance test.....	28
2.4 Ultra high performance fiber reinforced concrete	30
2.5 Physical and chemical properties of UHPFRC.....	34
2.5.1 Compressive strength.....	35
2.5.2 Tensile and Flexural Strength	37
2.5.3 Toughness test.....	39
2.5.4 Creep and shrinkage.....	40
2.5.5 Dynamic strength test	40

Table of Contents (continued)

	Page
2.5.6 Durability	41
2.5.6.1 Rapid chloride ion permeability.....	42
2.5.6.2 Porosity	42
2.5.6.3 Permeability test.....	43
2.6 Ductal (grey premix).....	45
2.7 Properties of Ductal with polyvinyl alcohol fibers	47
2.8 Properties of Ductal with steel fibers.....	48
2.8.1 Compressive strength test	48
2.8.2 Tensile and flexural test	50
2.8.3 Dynamic testing	51
2.8.4 Static and dynamic load test	53
2.8.5 Modulus of elasticity and Poisson's ratio	53
2.8.6 Rapid chloride penetration test	54
2.8.7 Shrinkage and creep	55
2.8.8 Freezing and thawing cyclic test.....	55
 III. EXPERIMENTAL PROGRAM	 57
3.1 Materials	57
3.1.1 Ductal grey premix	57
3.1.2 Quikrete® Non-Shrink Precision Grout	58
3.1.3 Precast concrete (High early concrete mix)	59
3.1.4 Mixing.....	60
3.2 Specimens	61
3.2.1 Modified ASTM G 109 samples.....	61
3.2.2 Full-size specimen of a section of bridge deck.....	67
3.3 Experimental test setup	76
3.3.1 Half-cell potential test.....	76
3.3.2 Linear polarization test	79
3.3.3 Visual observation	81
 IV. RESULTS AND DISCUSSION	 83
4.1 Modified ASTM G 109 sample	83
4.1.1 Precast concrete	83
4.1.2 Ductal with PVA fibers.....	86
4.1.3 Ductal with Steel fibers.....	89
4.1.4 Precast concrete + Ductal with PVA fibers	92
4.1.5 Precast concrete + Ductal with Steel fibers	95

Table of Contents (continued)

	Page
4.2 Uncracked large samples	99
4.2.1 Ductal PVA	100
4.2.2 Ductal Steel	105
4.2.3 Quikrete with PVA fibers	110
4.3 Cracked large samples	115
4.3.1 Ductal PVA Static (STA - 04)	116
4.3.2 Ductal Steel Static (STA - 02)	122
4.3.3 Ductal Steel Fatigue (FAT - 02)	127
4.3.4 Quikrete PVA Fatigue (FAT - 01)	132
4.3.5 Quikrete PVA Static (STA - 01)	138
4.4 Summary of results	143
 V. CONCLUSION AND FUTURE RECOMMENDATION	145
5.1 Conclusions	145
5.2 Recommendation for future works	146
 APPENDICES	148
A: Properties for Quikrete® Non-Shrink Precision Grout	149
B: Aggregate properties	151
C: Chloride penetration depth figures.....	153
D: Visual examination of the rebar (cores).....	163
E: Potential contour maps of large samples	182
 REFERENCES	192

LIST OF TABLES

Table	Page
2.1 Probability of corrosion according to half-cell potential reading	27
2.2 Mixture proportion design of UHPFRC	34
2.3 UHPC premix content	34
2.4 Characteristic strength of UHPFRC	35
2.5 Characteristic properties of Ductal	46
3.1 Ductal mix designs	58
3.2 Mix design of Quikrete Grout	58
3.2 Mix design of concrete.....	60
3.4 G109 sample details	62
3.5 Large sample details	69
3.6 Dates of casting large samples (cracked).....	71
3.7 Dates of casting large sample (uncracked)	76
4.1 LPR data of precast concrete sample 2	85
4.2 LPR data of Ductal PVA sample 2	87
4.3 LPR data of Ductal Steel sample 3	91
4.4 LPR data of PC + DP sample 1.....	94
4.5 LPR data of PC + DS sample 2.....	97
4.6 LPR data of Ductal PVA for sample 2 and 3.....	103
4.7 LPR data of Ductal Steel for sample 2 and 3.....	108
4.8 LPR data of Quikrete PVA for sample 2	113

List of Tables (Continued)

Table	Page
4.9 LPR test data for Ductal PVA Static (STA - 04)	120
4.10 LPR test data for Ductal Steel Static (STA - 02)	125
4.11 LPR test data of Quikrete PVA Fatigue (FAT - 01)	135
4.12 LPR data of Quikrete PVA Static (STA - 01)	141
4.13 Modified ASTM G109 Samples	143
4.14 Uncracked large samples	144
4.15 Cracked large samples	144

LIST OF FIGURES

Figure	Page
1.1 Corroding rebars at the bottom the slab of a bridge	6
1.2 Reinforcement corrosion cell.....	8
2.1 Cross section of precast adjacent box bridge	16
2.2 Pourbaix diagram for Fe-H ₂ O at 25°C	20
2.3 Schematic diagram showing spalling of concrete due to corrosion damage	24
2.4 Apparatus for half-cell potential method described in ASTM C 876 to measure surface potential associated with corrosion current	27
2.5 Half-cell potential contour map measured by CSE; Lower Elk Creek Bridge #2531, British Columbia	28
2.6 Linear polarization resistance curve	30
2.7 Stress strain curve	31
2.8 Dense filling image.....	32
2.9 Compression curve of Ductal	50
2.10 Displacement curve of Ductal	51
2.11 Creep curve of Ductal	55
3.1 G109 mold	62
3.2 Rebar used in G109 samples.....	63
3.3 Condition of rebar after wire brushing	63
3.4 Arrangement of rebar	64
3.5 G109 samples partially filled with concrete	65

List of Figures (continued)

Figure		Page
3.6	G109 samples completely filled.....	65
3.7a	G109 sample made of one material	66
3.7b	G109 sample made of two materials.....	66
3.8	After applying water proofing compound.....	67
3.9	Elevation of large sample.....	68
3.10	Arrangement of steel reinforcement in large samples	70
3.11	Cross section of large sample	70
3.12	Ductal mixture	72
3.13	Curing of the shear key	72
3.14	Loading machine.....	74
3.15	Large sample with a pond on top.....	74
3.16	Large samples (uncracked)	75
3.17	Copper/copper sulfate electrode and saturated Calomel electrode	77
3.18	Markings on large samples	78
3.19	Different sides of measurement	78
3.20	Arrangement of LPR measurements	80
3.21	Coring machine.....	82
4.1	HCP graph of precast concrete in wet cycle	84
4.2	HCP graph of precast concrete in dry cycle.....	84
4.3	Precast concrete rebar	86

List of Figures (Continued)

Figure	Page
4.4 HCP graph of Ductal PVA in wet cycle	86
4.5 HCP graph of Ductal PVA in dry cycle.....	87
4.6 Ductal PVA rebar.....	89
4.7 HCP graph of Ductal Steel in wet cycle	90
4.8 HCP graph of Ductal Steel in dry cycle.....	90
4.9 Ductal Steel rebar.....	92
4.10 HCP graph of PC + DP in wet cycle.....	93
4.11 HCP graph of PC + DP in dry cycle	93
4.12 PC + DP rebar	95
4.13 HCP graph of PC + DS in wet cycle.....	96
4.14 HCP graph of PC + DS in dry cycle	96
4.15 Sample A.....	97
4.16 Sample B	98
4.17 Sample C.....	98
4.18 Position of cores on the large uncracked samples	100
4.19 Bottom part of Ductal PVA interface	100
4.20 HCP graph of Ductal PVA in wet cycle	101
4.21 HCP graph of Ductal PVA in wet cycle	102
4.22 Top rebar of shear key (sample 1)	104
4.23 Top rebar of interface (sample 2).....	104

List of Figures (Continued)

Figure	Page
4.24 Top rebar of precast concrete (sample 3).....	105
4.25 Bottom part of Ductal Steel interface	106
4.26 HCP graph of Ductal Steel in wet cycle	107
4.27 HCP graph of Ductal Steel in dry cycle.....	107
4.28 Top rebar of sample 1	109
4.29 Top rebar of sample 2	109
4.30 Top rebar of sample 3	110
4.31 Interface of Quikrete PVA	111
4.32 HCP graph of Quikrete PVA in wet cycle	112
4.33 HCP graph of Quikrete PVA in dry cycle	112
4.34 Top rebar of sample 1	114
4.35 Top rebar of sample 2	114
4.36 Top rebar of sample 3	115
4.37 Bottom surface of Ductal PVA Static.....	117
4.38 HCP graph of Ductal PVA in wet cycle	118
4.39 HCP graph of Ductal PVA in dry cycle.....	118
4.40 Top rebar of sample 1	121
4.41 Top rebar of sample 2	121
4.42 Top rebar of sample 3	122
4.43 HCP graph of Ductal Steel in wet cycle	123

List of Figures (Continued)

Figure	Page
4.44 HCP graph of Ductal Steel in wet cycle	123
4.45 Top rebar of sample 1	126
4.46 Top rebar of sample 2	126
4.47 Top rebar of sample 3	127
4.48 Bottom surface of Ductal Steel (FAT - 02)	128
4.49 HCP graph of Ductal Steel in wet cycle	129
4.50 HCP graph of Ductal Steel in dry cycle.....	129
4.51 Top rebar of sample 1	131
4.52 Top rebar of sample 2	131
4.53 Top rebar of sample 3	132
4.54 Bottom surface of Quikrete PVA Fatigue sample	133
4.55 HCP graph of Quikrete PVA in wet cycle	134
4.56 HCP graph of Quikrete PVA in dry cycle	134
4.57 Top rebar of sample 1	136
4.58 Top rebar of sample 2	136
4.59 Top rebar of sample 3	137
4.60 Bottom surface of Quikrete PVA Static (STA - 01)	138
4.61 HCP graph of Quikrete PVA in wet cycle	139
4.62 HCP graph of Quikrete PVA in dry cycle	139
4.63 Top rebar of sample 1	141

List of Figures (Continued)

Figure	Page
4.64 Top rebar of sample 2	142
4.65 Top rebar of sample 3	142

CHAPTER ONE

INTRODUCTION

1.1 Introduction

Corrosion of steel bridges, concrete bridges with embedded steel reinforcement and concrete pavements with embedded steel reinforcement has a significant impact on the performance of transportation infrastructure. In a recent study, the total direct cost of corrosion was estimated to be \$276 billion per year out of which the cost associated with the transportation infrastructure is 16.4 % (Yunovich, Thompson, and Virmani, 2005). Highway bridges are a critical component of the transportation infrastructure since the failure or load posting of a bridge can have a dramatic influence on the functionality of a transportation route. The funds required for repairing or replacing a bridge structure and the losses associated with the posting or temporary closure of traffic lanes during construction or maintenance of a bridge can have a substantial cost to a community. Nearly \$8.3 billion is the estimated average annual direct cost of corrosion for highway bridges. Reinforced concrete bridges account for 40% of all the bridges constructed in the United States since 1950. There are 543,019 reinforced concrete and steel bridges out of which 78,448 are classified as structurally deficient leaving rest of them in a working condition subjected to regular maintenance (Yunovich, Thompson, and Virmani, 2005). Most of these reinforced concrete bridges are constructed of precast concrete adjacent box girder bridges. The use of precast concrete elements in the form of adjacent box girder has been popular since 1950s due to its numerous advantages over the other types of bridges such as: ease and rapid speed of construction because of the elimination of

concrete formwork, a shallow superstructure depth necessary for vertical clearance, low construction cost, hollow portions inside the box girders that reduce the self-weight of it and helps to carry electrical, communication cable and various pipe lines (inside the hollow space in case of large box girders and are hung to the bottom of the soffit beam in case of small box girders) , and it offers high torsional stiffness (Hanna, Morcous, and Tadros, 2009).

These bridges have been in service for some time with a reasonable performance record except for one major issue. Occurrence of cracks on the grouted filler (shear key) joint which is used to provide continuity between two adjacent box girders together, results in the reflective cracks on the wearing surface. The cracks can form either during the construction or during the usage of the bridge. During the construction, the cracks are induced between the interface of the precast elements and the shear key due to the bridge deck being subjected to a large amount of dead loads by the placement of raw materials, heavy cranes and equipment required during the construction process which results in the subjection of heavy dead loads to the bridge deck leading to unaccounted vibrations resulting in the formation of undesirable cracks between the two different materials. While the bridge is in operation and used by the ongoing traffic the cracks are formed on the wearing surface due to heavy axial loads (running wheel load) and irregular maintenance of the wearing surface of the deck causing the majority of the cracks to form directly over the interface of precast elements and the shear key present beneath the wearing surface since they might have already cracked and weakened during the construction process. The cracking leads to leakage and seeping of water containing

corrosive agents, like chloride, magnesium and sulfate ions and oxygen to the bottom and sides of the girder which causes the embedded steel reinforcements and the pre-stressing strands to corrode and thus reduces the structural and serviceable strength of the entire bridge structure. The first failure of this system type was recorded in Indiana in 1970s. Other similar failures include a bridge in Cuyahoga County, Ohio that failed in 1989 and a bridge on Lake View Drive in Pennsylvania that failed in 2005. These three failures have the same commonality of cracking on the bridge deck resulting in ingress of chloride-laden water corroding the steel reinforcement (Naito, Jones, and Hodgson, 2011).

The structural elements of a bridge can be primarily classified into three sub-systems: foundation, sub-structure and super-structure. The foundation (footers, caissons, piles) provides the load transfer from the sub-structure to the ground supporting the bridge structure. Abutment walls, columns and column bents, piles and pile caps are considered to be the sub-structure and support the primary elements of the bridge. The structural elements and decks within the super-structure tend to be more vulnerable to corrosion due to greater likelihood of cracking of the concrete. The exposure of deck elements to the atmosphere supplements the water and oxygen needed to instigate the corrosion of the embedded reinforcement. The typical concrete cover depth provided for the reinforcement of the deck (1 to 2 inches) is less as compared to the cover provided for foundations and columns (3 to 4 inches). The foundation and the sub-structure elements of the bridge are usually subjected to direct compression loads whereas the bridge deck in complementary with the former is also subjected to flexural, tension and shear loads at

the center of the span and at the supports resulting in cracks at the soffit of the beam providing an entrance to the corrosive agents to initiate the process of corrosion in the tension reinforcement (at the bottom of the slab) as shown in Figure 1.1. Precast concrete has replaced the conventional cast-in-place concrete due to the overwhelming advantages that the former offers over the latter; speedy construction since there is no need for placing formworks, grading the aggregates, mixing of ingredients, curing of concrete and also the cost is reduced since there will be no need for employing foremen to carry out the above operations. But still precast concrete has some of its own disadvantages over cast-in-place concrete no cost have to be incurred on the transportation of the prefabricated elements safely from the casting yard to the site and there will be no need to expend capital on hiring expert technicians for installation of these prefabricated elements, the contractor need not worry about careful handling and placing of these elements on the construction site, but the biggest advantage lies in the fact that the structure is monolithic with minimum joints between column, beams and slabs since all of them will be casted at the same time. Structures made up of precast concrete elements have plenty of segmental joints in the form of shear key and closure pours which are required to maintain the continuity of the structure so that the structure can behave as monolithic as possible.

Usually the weakest point of any type of structure can be defined as the connections or the joints that exists in them and therefore careful design and due considerations are given to it while designing and construction. Similarly in the case of precast adjacent box girders the weakest point can be considered as the shear key which

helps to join the two adjacent precast elements together so that there is a perfect bond between them to serve as a single unit for maintaining a continuity of the bridge. The cracks are formed usually on the interface of the shear key and the precast elements allowing the ingress of water through them. In the earlier construction process high precision grouting materials were used as filler joint to act as a shear key between the two precast elements since they possessed high and improved properties than precast concrete. But this material was found to be permeable leading to leakages and formation of surface and bonding cracks making it obsolete and lead to the development of Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) which is a material that possesses properties far more superior than the ordinary and high performance concrete and therefore replacing the (high strength pressure fluid) grouting material as a shear key. UHPFRC is a fairly new material introduced to the construction industry nearly a decade ago and hence extensive amount of research on its physical and chemical properties are being carried out by the government agencies and the construction companies to analyze its effectiveness in counteracting the problems faced by the conventional grouting materials. This technology in the US is relatively new that claims to meet the shortcomings of many types of concrete through its high strength to weight ratio, high ductility and enhanced durability properties. But still, to gain acceptance by designers, contractors, precasters, and owners, this material needs to be tested according to ASTM International and AASHTO standards, and new practices must be developed. Other parameters like curing methods and practices, age of the specimen on strength, resistance to environment etc. need to be investigated (Ahlborn, Harris, Misson, and Peuse, 2011).

Extensive amount of research has been carried out on the compressive, tensile, flexural, permeability, shrinkage, curing and microstructure of UHPFRC which have explicitly surpassed the properties of all its predecessor materials with realistic models and values. Permanent values and relevant standards of the entire above aforementioned properties have been established by the results obtained from the experiments conducted on UHPFRC.

Figure 1.1 Corroding rebars at the bottom the slab of a bridge

1.2 Corrosion of steel reinforcement bars

Corrosion can be defined as the destructive result of a chemical reaction taking place between the metal and its surrounding environment. A metal in its original form (minerals and ores) is fairly weak and incapable of any usage therefore it is altered by the process of metallurgy and converted into a new metal which possesses properties like toughness and high strength and thus they can be put into numerous applications enhancing human life. Thus through the process of corrosion the metal tries to regain its original form. Transverse and longitudinal steel reinforcement bars are present in the shear key of the bridge deck from the perspective of the bridge design. It is imperative for the safety of the user of a bridge structure to carryout regular maintenance and experimental tests which can determine the probability of corrosion of the embedded rebars and also to measure the amount of corrosion at the interface of the embedded reinforcement passing through different materials (from precast concrete elements into the shear key which is made up of UHPFRC). When the steel is embedded in the concrete, the high alkaline nature ($\text{pH}=14$) of concrete forms an oxide layer around the periphery of the rebar that acts as passive film to protect the bar from corrosion. The protective film prevents iron cations (Fe^{++}) from entering into solution in the electrolyte (concrete mass) and acts as a barrier to prevent oxygen anions (O^-) from coming in contact with the steel surface. However, this protective film is not invincible and thus can be disrupted either by a reduction in the pH of the pore solution due to carbonation, or by the penetration of aggressive ions like chlorides at the steel-concrete interface (Al-Amoudi and Maslehuddin, 1993). The corrosion of steel in concrete is essentially an

electrochemical process, where at the anode, iron, is oxidized to iron ions that pass into solution and at the cathode oxygen is reduced to hydroxyl ions (Yoo, Park, Kim, and Chung, 2003). The surface of the corroding steel reinforcement bar which is embedded in the concrete mass functions as a mixed electrode that is a composite of anodes and cathodes electrically connected through the body of steel itself, upon which coupled anodic and cathodic reactions take place while concrete pore water functions as an aqueous medium, i.e., a complex electrolyte. Therefore, a reinforcement corrosion cell is formed as showed in Figure 1.2 (Ahmad, 2003).

Figure 1.2 Reinforcement corrosion cell

Corrosion of steel in concrete basically occurs in three steps; depassivation, propagation, and final state. Depassivation is the process of losing the oxide layer around the periphery of the rebar which is formed due to the alkaline nature of concrete then follows the expansion of the corrosion on the surface area of steel, causing it to de-bond with the surrounding concrete layers. In the final state the expansive corrosion products

(rust) causes the concrete to crack and spall by exceeding its internal tensile stress limits (Ahmad, 2003).

The first step to identify the ongoing corrosion is the visual inspection of the structure and then to use the various electrochemical devices available in the market to find the extent and the rate of corrosion. Quality concrete is able to prevent the embedded reinforced steel bar from the potential of corrosion in three ways. First, hydration products of cement in concrete forms a high alkaline pore solution environment, where the passivated film covering the steel surface remains chemically stable enough to protect reinforcing steel from corroding. Second, quality concrete usually possesses low porosity and permeability, which greatly minimizes the penetration of corrosion-induced agents (such as chloride, carbon dioxide, oxygen, moisture, etc.) through the porous concrete mass. Third, the high electrical resistivity of quality concrete restricts the corrosion by reducing the electrical current flow between the anodic and cathodic sites (Chen and Mahadevan, 2008).

Factors affecting corrosion of steel in concrete may be classified into two major categories; internal and external factors. The internal factors mainly consist of environmental parameters; availability of oxygen and moisture at rebar level, relative humidity and temperature, cement composition, impurities in concrete ingredients, water cement ratio. The external factors consist of stray currents, bacterial action (Ahmad, 2003). According to the Ueli M. Angst who carried out an experiment in which reinforcement steel was embedded in six different concrete mixes and exposed to chloride solutions in cycles of wet and dry periods. Pitting was observed to be formed at

the backside (respect to casting direction) of the reinforcement embedded in the concrete. The concrete in contact with the rebar on the front side was whitish in color while at the backside was grey in color indicating that there was a good bond at the front side as compared to back side of the rebar, may be due to bleeding, plastic settlement or higher w/b ratio. Corrosion occurred at the backside of the rebar even though higher chloride concentration was present at the front side. Hence he concluded that the most dominant influencing parameter is the steel-concrete interface with respect to the corrosion performance of embedded steel bar in concrete environment (Angst, Elsener, Larsen, and Vennesland, 2011).

1.3 Objectives

Very little research has been carried out on the aspect and the extent of corrosion of the embedded reinforcement bar and pre-stressing strands in UHPFRC which is the leading cause for failures of bridge girders. Ample amount of research has been done on the corrosion of reinforcement embedded in ordinary concrete and on the individual properties of materials like concrete and UHPFRC but in reality these materials act as a single unit in a bridge deck. There are number of bars passing between two precast concrete elements having a shear key or a closure pour between them and hence no research have been conducted so far by considering the combined effect of precast concrete and the shear key from the view point of strength and serviceability of the bridge. UHPFRC and precast concrete are vastly different from each other hence the reinforcement bar passing through both of them may undergo variable damaging changes along its length. No such research has been undertaken which evaluates the condition of

the embedded reinforcement passing through different medium of materials on the aspect of strength and durability in order to predict and examine the structural behavior of precast adjacent box beam girder bridges. Extensive amount of research has been carried out on the corrosion of steel reinforcement embedded in concrete, while negligible or practically research has been carried out on the corrosion of the rebar embedded in UHPFRC. Properties like compressive, tensile and flexural strength, shrinkage, curing, modulus of elasticity and rupture have been studied in detail and relative standards have been established for the design and construction purposes. But its performance with embedded reinforced bars which includes properties like bond strength, workability, and corrosion of reinforcement have seldom been tested or researched upon. The following are the objectives of this research;

- To identify the regions of active and passive corrosion of the embedded reinforcement with the help of non-destructive electrochemical techniques.
- To investigate the extent of corrosion of the rebar by measuring the corrosion current with half-cell potential and linear polarization resistance technique.
- To investigate the reliability of the above electrochemical technique in the prediction of corrosion of rebar embedded in UHPFRC.
- To observe the corrosion pattern with respect to cracks formation taking place in samples made of different materials.
- To identify the performance of various materials with respect to the corrosion undergone by the rebar.

1.4 Summary of approach

Ductal grey premix an ultra-high performance concrete manufactured by LaFarge, Non-Shrink Precision Grout manufactured by Quikrete and precast concrete mix design provided by Metromont (Greenville, SC) were used as primary materials in the casting of all samples. Steel and polyvinyl alcohol (PVA) fibers were used to produce fiber reinforced concrete and #4 steel deformed bars were used as longitudinal and transverse reinforcement. The mix designs are similar to the ones used in the actual construction of a bridge. Utmost care was taken during the mixing of the materials and casting of the specimens to minimize the variability of the results.

All the samples were subjected to 3% NaCl solution on the top surface of the specimens for a period of two weeks as a wet cycle and then followed by two weeks of dry cycle (without direct exposure to the solution). Modified ASTM G 109 samples were prepared to serve as control specimens from following the procedure of a standardized test. Full scale bridge deck samples which were structurally tested and had existing cracks within the precast concrete, within the shear key and in the interface of these surfaces, where also exposed to the 3% NaCl solution. Similar uncracked specimens were also tested for comparison to the other specimens.

All the samples were subjected to cyclic wet and dry period of two weeks for a total period of 44 weeks. Half-cell potential (HCP) measurements were taken once at the end of each period of G 109 and large samples. After determining the most and the least corrosive samples of G 109 and points on the large samples with the help of potential contour charts formed by measuring HCP on a number of selected points on the samples

and then electrochemical techniques such as linear polarization test and cyclic polarization test was carried out to estimate the rate of corrosion. The results so obtained were compared with condition of the cores extracted from the large samples at the end of the experiments to observe the condition of the rebar in order to have a visual confirmation on the results obtained from the electro-chemical techniques. Based on the results obtained from the above tests on the specimens, valuable information was obtained which helped in drawing conclusions regarding the performance of UHPFRC in conjunction with reinforcement bars when subjected to salt solution in the presence and absence of surface cracks.

1.5 Outline of thesis

Chapter 2 is concerned with the complete background report of the above problem. The conditions and research carried out on the bridges, corrosion of the rebar in concrete, the techniques adopted to detect and measure the corrosion of the embedded reinforcement. Literature review of research carried out on UHPFRC and its properties along with relevant tests. Chapter 3 discuss about the materials used in this research along with the properties, type of samples prepared for the research and all the experiments and various electro-chemical techniques used in this research to predict and examine the condition of the rebar embedded in precast concrete and UHPFRC. Chapter 4 discusses the results obtained from the experiments in detail. Scrutinizing and analyzing the results of the test with that of core samples obtained from the samples at the end of the testing period. Chapter 5 deals with the conclusions of the research along with

the recommendations and future works that can be carried out on the same and relevant topics.

CHAPTER TWO

BACKGROUND

Reinforced concrete structures can possess numerous advantages over a steel frame, plain concrete, masonry and timber structures, such as properties like; higher durability, better mechanical performance, less permeable to flowing liquids, increased resistance to abrasion, atmosphere and chemical attacks. Therefore, all these characteristics concludes that reinforced concrete structures are more efficient, reliable and have a longer service life than any other type of structures for providing shelter to human life as of present. However, corrosion of steel reinforcement embedded in concrete is the main deterioration mechanism for such structures.

Corrosion of the rebar is the one of the most disastrous chronic problems occurring in reinforced concrete structures. Once initiated it exists in the structure till the end of its service life. Corrosion is an electrochemical reaction which occurs when the metal (rebar) is exposed to aggressive species such has chloride ions. To overcome this problem many investigations have been carried out to minimize the corrosion related issues in the reinforced concrete so that the structure maintains its integrity. One of the approaches is to develop less permeable concrete materials to reduce the diffusion of the corrosive species. High Performance Concrete (HPC), Reactive Powder Concrete (RPC), Ultra High Performance Concrete (UHPC), and Ultra High Performance Fiber Reinforced Concrete (UHPFRC) are some of the examples of such materials.

In this Chapter, the information on: the precast concrete adjacent box bridges, their performance and utilities, corrosion of embedded reinforcement in concrete, the

most common corrosion measuring techniques, UHPFRC and its properties and properties of Ductal grey premix.

2.1 Bridges

According to the National Bridge Inventory database there are nearly 600,000 bridges existing in the United States half of which were constructed between the periods of 1950 to 1994. There are approximately 543,000 concrete and steel bridges out of which 78,488 are declared as structurally deficient leaving rest of the bridges on a regular basis to be maintained. After diligently including all the necessary facts an estimated figure of \$8.3 billion dollars for the average annual direct cost of corrosion for the highway bridges was worked out (Yunovich et al., 2005). This research is focused on precast concrete adjacent box beam girder bridges as shown in Figure 2.1, sometimes they are also referred as hollow precast box bridges due to its cross sectional geometry.

Figure 2.1 Cross section of precast adjacent box bridge
(Clay Naito 2011)

Precast concrete adjacent box girder bridges were constructed in large numbers since 1950's due to the technique of Accelerated Bridge Construction (ABC). This was widely accepted in most parts of northeastern United States due to its innovative

planning, reduced cost and rapid speed of construction. Hollow concrete bridge decks with embedded reinforcement were cast at casting yards in bulk quantity which resulted in decrease of time in construction and the cost of the bridge. These bridges consist of multiple precast concrete girders that are joined against each other to form the bridge deck. They were post-tensioned transversely with steel strand wires at the site and the adjacent slabs were connected using full or partial depth shear keys which consist of high strength grouting material. The system of load transfer of moving traffic relied on the strength and stiffness of the shear key and the post-tensioned strands (Hanna et al., 2009; Yunovich et al., 2005).

Box beam girder bridges have performed well but the major problem arising is the corrosion of pre-stressing strands and the embedded reinforcement in the bridge deck due to the ingress of chloride ions in the bridge deck which is caused due to the following reasons:

- 1) Longitudinal cracks on the shear key in the grouted joints and on the interface between the grout material and precast concrete elements arising from temperature gradients, dynamic loads (imparted from the moving road traffic), and static loads (imparted during the construction of the bridge from the stationary heavy duty cranes and other equipment's like air compressor, wheel rollers, concrete pumping machine and vibrators.
- 2) A lack of adequate transverse force within the prestressing strands due to poor workmanship results in differential movements between the adjacent precast girders leading to un-anticipated crack patterns on the deck.

- 3) Presences of open vent holes in beams with cardboard form voids collect the water and clog the drain holes resulting in pooling of stagnant water in the voids present on the deck.
- 4) The use of snowplows to remove snow that accumulates on the bridge deck leads to openings on the coated or uncoated bridge decks due to the physical contact (abrasion) between the steel blades and the asphalt surface.

The chloride analysis of a corroding bridge deck has revealed that the chloride content in concrete around the cracks exceed the critical Chloride Threshold Level (CTL) within the first year of its construction, hence the reinforcing steel must be protected from corrosion at the time of its construction (Darvin, Browning, O'Reily, Locke Jr. and Virmani, 2011). Presence of cracks on the bridge decks provides a free path to the chloride ions to reach the steel reinforcements (tensioned and non-tensioned) and cause corrosion. Use of de-icing salts, close proximity to coastal salt water, salt sprays from recreational water vehicles are some of the sources responsible for the presence of chloride ions on the bridge decks (Naito et al., 2011). Chloride ions alone are innocuous to rebars but along with oxygen and moisture it can initiate the corrosion process as soon as they come in contact with the reinforcement surface leading to de-bonding between steel and concrete forcing the reinforce concrete to forfeit its tension carrying capacity. Such condition jeopardizes the durability and structural behavior of the precast adjacent box beam girders and due to the clogged drains the water starts getting collected in the hollow precast boxes which causes the loaded girders to carry additional load that exceeds the design load (Hanna et al., 2009).

After careful study and examination of the box beam failures that have occurred in the past following suggestions are put forward to improvise the current design and construction practices of such bridges to prevent their failures due to the action of corrosion;

- 1) Post-tensioned high strength ties should be incorporated in the design since they are capable of increasing the tensile carrying capacity of the concrete in conjunction with epoxy coated reinforcement bar which can increase the service life by decreasing the probability of corrosion.
- 2) High strength grouting material in the form of ultra-high performance fiber reinforce grout is used to fill the shear key instead of non-shrink grout.
- 3) The interface of precast elements to be grouted is sand blasted to improve its bond strength.
- 4) The depth of concrete cover at the soffit of the beam and the number of tendons in the transverse directions are increased.
- 5) Shear key is provided to the full depth of the girder cross-section.
- 6) New and improved design methodology is provided by the organization of Precast/Prestressed Concrete Institute (PCI) in their published journals and articles (Hanna et al., 2009).

2.2 Corrosion and its mechanism

Concrete gives corrosion resistance to steel reinforcement because it provides both a physical barrier and chemical protection. Steel is thermodynamically unstable in atmosphere and tends to revert to a lower energy state such as an oxide or hydroxide by

reaction with oxygen and water. Concrete that is not exposed to any external influences usually has a pH between 12.5 and 13.5 (Hansson 1984). As shown in the Pourbaix diagram Figure 2.2, this defines the range of electrochemical potential and pH, for H₂O-Fe system in the alkaline environment and at the potentials normally existing in the concrete, a protective passive layer forms on the surface of steel. This layer is an ultra-thin (< 10 nm), protective oxide or hydroxide film that decreases the anodic dissolution rate to negligible levels.

Figure 2.2 Pourbaix diagram for Fe-H₂O at 25°C
(Pourbaix 1974)

The protective nature of this layer can be reduced and the result would be active corrosion of steel in concrete. In North America, chloride ions is the major factor that can break the passive film on the surface of steel and initiate corrosion and the mechanism will be discussed in the next sections. A localized breakdown of the passive layer occurs

when sufficient amount of chlorides reach reinforcing bars, and the corrosion process is then initiated. There are three theories about the chloride attack (ACI 222, 1996):

1. Penetration of chloride ions to the oxide film on steel through pores or defects in the film is easier than the penetration of other ions.
2. Chloride ions are adsorbed on the metal surface in competition with dissolved O_2 or hydroxyl ions.
3. Chloride ions compete with hydroxyl ions for the ferrous ions produced by corrosion and a soluble complex of iron chloride forms which can diffuse away from the anode, destroying the protective layer of $Fe(OH)_2$ permitting corrosion to continue.

The chloride induced corrosion of the steel reinforcement results in the formation of expansive corrosion products in the concrete which causes cracking and spalling of the concrete (Yunovich et al., 2005). The volume of corrosion products is greater than that of original steel bar. This results in expansive stresses around the corroded steel bar exceeding the tensile strength limit of surrounding concrete mass resulting in cracking, spalling of concrete cover and loss of bond between steel and concrete (Abosrra et al., 2011; Abosrra, Ashour, and Youseffi, 2011). A crack facilitates the entry of chloride ions, moisture and oxygen to the reinforcements (Charron, Denarie, and Bruhwiler, 2007).

Corrosion of reinforcement is controlled by increasing the concrete cover and crack width limitations by choosing adequate steel bar spacing which will result in permissible stresses in reinforcement. The availability of ultra-high performance fiber

reinforce concrete (UHPFRC) can be utilized in resisting and curtailing the occurrence of corrosion in the vulnerable parts of any reinforced structures. This can be achieved by placing this material in the parts of bridge which have high probability of getting infected of corrosion especially in the bridge super-structure elements like parapet walls, wearing surface of bridge deck, drain and vent holes. Due to its dense micro-structure and low porosity it practically becomes impregnable for the even for the stagnant chloride laden water to penetrate it (Charron et al., 2007). In order to reduce corrosion, selection of concrete with resistance to chloride penetration is very essential to maintain the integrity of reinforced concrete structures (Shi, Deng, and Xie, 2006).

2.2.1 Kinetics and thermodynamics of corrosion

Corrosion is an electrochemical reaction which consists of anodic and cathodic half-cell reactions. Micro-cell corrosion is the term given to the situation where active dissolution and the corresponding cathodic half-cell reaction take place at adjacent parts of the same metal part. For a steel reinforcing bar (rebar) in concrete, this process always occurs in practice. The surface of the corroding steel can act as a mixed electrode containing both anode and cathode regions which are connected by the bulk steel. Macro-cells corrosion can also form on a single bar exposed to different environments within the concrete or where part of the bar extends outside the concrete. In both cases, concrete pore solution functions as an electrolyte. Figure 1.2 shows a schematic illustration of corrosion in reinforcing concrete.

For steel embedded in concrete, based on the pH of the concrete (electrolyte) and presence of aggressive ions, the following would be the possible anodic reactions [Ahmed 2003, Hansson 1984]:

The possible cathodic reactions depend on the availability of O_2 and on the pH near the steel surface. The most likely reactions are as follows (Ahmed 2003, Hansson 1984):

The corrosion products occupy a greater volume than the steel itself, and this causes an internal expansion and stress. The stress can destroy the concrete and expose the steel to more aggressive factors. Figure 2.3 shows a schematic illustration of a damaged concrete by corrosion of reinforcement steel.

Figure 2.3 Schematic diagram showing spalling of concrete due to corrosion damage
(Corrosion-club, 2004)

2.2.2 Influence of cracks on corrosion

Reinforced concrete structures were considered to be a durable and maintenance free structures; however, concrete is capable of developing cracks on its surface due to drying shrinkage, alkali silica reaction, thermal effects, and temperature gradient making it vulnerable to deteriorate before the end of its service life. The cracks can lead to the entry of water containing corrosive species inside the concrete causing the reinforcements to corrode (Tsukahara and Uomoto, 2000). Smaller the frequency of cracks the lesser is the amount of corrosion of steel in concrete irrespective of all other parameters (Arya and Ofori-Darko, 1996). The corrosion cracks can also form on the concrete surface due to the formation of corrosion products which are in excessive volume than the base metal (steel reinforcement) causing the surrounding mass to exceed the tensile limit of the material (concrete). The appearance of cracks on the concrete surface provides a free

entrance to the corrosive species to travel inside the concrete (Avelano and Ortega, 2011). Reinforced concrete structures having surface cracks, located in places that receive snowfall can also undergo the freeze-thaw effect which leads to deeper cracks that might reach the surface of the rebar, and consequently, act as the free path to aggressive species. Corrosion of steel reinforcement results in decrease in its cross-sectional area and also deteriorates the bond between the steel reinforcement and the concrete which leads to the degradation of the entire structure (Kato, Kato, and Uomoto, 2005). A crack has the ability to accelerate the dynamic action of corrosion and thus shortens the life of the structure. The corrosion of steel in cracked concrete is localized as compared to the corrosion in un-cracked concrete (Otieno, Alexander, and Beushausen, 2010).

Most of the research and experiments to establish a relationship between crack and corrosion have used or focused on unloaded or statically loaded cracks. However in actual practice reinforced concrete structures are subjected to variable loadings, hence the cracks in such structures open and close with the application of load which could influence the corrosion of the steel. This was studied by Jaffer and Hansson, through their experiments they observed that the corrosion had occurred on the rebar only at places which were intercepted by cracks, the type of loading has less significance on the process of corrosion than other parameters like exposure conditions and type of concrete (Jaffer and Hansson, 2008). It is imperative to remember that concrete always has cracks (in one form or the other) on its surface which have an immense impact on the corrosion of rebar

embedded in it. The presence of cracks also avails the access to oxygen and moisture which both are required for the corrosion process to occur.

Experiments have shown that in a specimen with multiple cracks, corrosion initiates earlier at the major (wider) cracks. Defining a crack width limit to minimize corrosion is a difficult task. However several building codes like ACI 224 (2009) and ACI 318 (2009) limit crack widths to 0.4 mm (0.016 in.) (Bhaskar, Gettu, Bharatkumar, and Neelamegam, 2011).

2.3 Corrosion measuring techniques

There are many methods available for evaluating the corrosion of the embedded reinforcement bar in concrete. The questions arising for selecting a suitable technique for carrying out the above operation are as follows; technique to be used in the field or the lab, the amount of accuracy required, the amount of time and funds available for the tests, and should it be destructive or non-destructive. Below are the details of the techniques that were used in this investigation which are the most common corrosion evaluation methods in steel reinforced concrete structures.

2.3.1 Half-cell Potential Test (HCP)

The half-cell potential technique is the most widely used technique of corrosion measurement of the steel rebars in concrete. It was introduced in the 1970s by Richard F. Stratfull in North America and by the Danish Corrosion Centre in Europe (Stratfull 1972). In 1980, the C 876 “Standard Test Method for Half-cell Potentials of Uncoated Reinforcing Steel in Concrete” test was approved as a standard by ASTM. This

technique is based on measuring the electrochemical potential of the steel rebar with respect to a standard reference electrode placed on the surface of the concrete and can provide an indication of the corrosion risk of the steel. The suggested reference electrode by ASTM is a copper/copper sulfate electrode (CSE). A wet sponge should be placed between the electrode and the concrete to provide a low electrical resistance i.e. good contact between the electrode and the concrete. Figure 2.4, shows the basics of half-cell potential measurement and Table 2.1 shows the interpretation of half-cell potential readings taken from copper/copper sulfate reference electrode.

Figure 2.4 Apparatus for half-cell potential method described in ASTM C 876 to measure surface potential associated with corrosion current

Table 2.1 Probability of corrosion according to half-cell potential reading

Half-cell potential reading vs. Cu/CuSO ₄	Corrosion activity
More positive than -200 mV	90% probability of no corrosion
Between -200 and -350 mV	An increase probability of corrosion
More negative than -350 mV	90% probability of corrosion

The most common way of presenting the half-cell potential data is plotting the potential distribution or potential mapping contour as shown in Figure 2.5.

Figure 2.5 Half-cell potential contour map measured by CSE; Lower Elk Creek Bridge #2531, British Columbia
(Gepreags and N.A.Cumming, 2006)

2.3.2 Linear Polarization Resistance Test

Linear polarization resistance (LPR) is a non-destructive technique used to evaluate the rate of corrosion in the embedded steel. LPR is used to measure an average corrosion rate over the whole steel surface area (Otieno et al., 2010). The theory of LPR is based on the relationship between the half-cell potential of a piece of corroding steel and an externally current applied to it, i.e., the corrosion rate is proportional to the applied current divided by the potential shift. Methods based on linear polarization resistance are the most suitable to be applied on-site according to (Feliu, Gonzalez, and

Andrade, 1996). An LPR test uses a non-corroding counter electrode (stainless steel) and a reference electrode for establishing a polarization curve by imposing a range of potentials on the metal which is corroding and measures the corresponding corrosion currents by using a potentiostat (Darvin, Browning, O'Reily, Locke Jr. and Virmani, 2011). LPR has the advantage of providing a direct measurement, of the corrosion rate of steel (Gowers and Millard, 1993). A range of -10 mV to 10 mV or -20 mV to 20 mV potential is applied to the specimen and the corresponding current response is recorded. Potential can be applied as a constant pulse (potentiostatic), or a potential sweep (potentiodynamic) (C. M. Hansson, A. Poursaee and S. J. Jaffer 2007). The above-mentioned potential is preferable because it provides an assurance on the part of maintaining the condition of linearity of the current vs. the voltage curve as shown in Figure 2.6. The slope of the linear region is the polarization resistance, R_p . The total corrosion current density is obtained by using the following relationship;

$$I_{\text{corr}} = \frac{B}{R_p}$$

Where:

I_{corr} = corrosion current (A)

B = Stern – Geary equation typically taken as 26 mV during active corrosion

R_p = Slope determined from the polarization curve (kilohms*cm²).

Figure 2.6 Linear polarization resistance curve
(Amir Poursaei, 2011)

2.4 Ultra High Performance Fiber Reinforced Concrete (UHPFRC)

Since last two decades, astonishing advancements has been made in the field of concrete technology. Materials like densified small particles concrete (DSP), macro defect free concrete (MDF) and reactive powdered concrete (RPC) have been marketed as high performance concrete in various countries. One of the latest advancements in concrete technology is ultra-high performance fiber reinforced concrete (UHPFRC). In 1990's UHPFRC was developed by the addition of supplementary materials, elimination of coarse aggregates, very low water/binder ratio (less than 0.25), application of super-plasticizer, addition of fine steel fiber reinforcement, heat curing and application of pressure before and during setting (Gao, Molyneaux, and Patnaikuni, 2008). It is a special cement based material which behaves like a low porosity ceramic material and is densely

packed that exhibits increased mechanical performance due to high stress and strain relationship, as shown in Figure 2.7, and superior durability compared with normal- and high-strength concretes (Ahlborn et al., 2011)(Corinaldesi and Moriconi, 2012)(Barnett, Lataste, Parry, Millard, and Soutsos, 2010) as thus, it can be described as a high strength, ductile, and sustainable construction material formulated by combining Portland cement, silica fume, fine washed/sieved sand, super-plasticizer, water, and steel fibers (Nematollahi, Saifulnaz, Jaafar, and Yen, 2010).

Figure 2.7 Stress strain curve
(Valeria Corinaldesi 2011)

UHPFRC is developed on the basic principle of a material that has minimum weakness as compared to ordinary concrete and is founded on the following four principles that can be summarized as follows:

- 1) Optimized granular packing which improves homogeneity of the mix and makes a very ultra-dense matrix as shown in Figure 2.8.
- 2) Extremely low water cement ratio which reduces the amount of pores and capillaries.

- 3) Inclusion of very high strength micro-fibers (steel fiber) which enhances tensile strength and ductility.
- 4) Steam curing which accelerates early and drying shrinkage and improves volumetric stability and creep (Nematollahi et al., 2010).

Figure 2.8 Dense filling image

Resulting matrix is extremely impermeable due to the sealing of pores, providing resistance to aggressive agents which allows chloride diffusion of only 0.02×10^{-11} sf/sec (Perry, Moore, and Bierwagen, 2006).

The typical mixture proportion design of UHPFRC is shown in Table 2.2 and Table 2.3 displays the UHPC premix contents. The only type of aggregate present in it is fine silica sand with particle sizes in the range 200–600 μm . To enhance the tensile strength and increase ductility of the cast sections, usually short straight steel fibers are added to the mixture (Barnett et al., 2010). The tensile strength of the steel fibers is in the range of 850–2000 MPa (123–290 ksi). The increase in the amount of steel fibers in the mix also increases the toughness of UHPFRC. The presence of steel fibers increases its

peak compressive strength and enhances its post peak behavior (Hassan, Jones, and Mahmud, 2012) (Skazlic and Bjegovic, 2009) (Kazemi and Lubell, 2011). A few physical properties of the UHPFRC have already been investigated such as the compressive strength, which is consistently attained above 270 MPa (30 ksi) regardless of thermal treatment, flexural strength of 25 to 30 MPa (3.6 to 4.4 ksi), Poisson's ratio of 0.21, very high resistance to freezing and thawing cycles and negligible permeation to chloride ions (Ahlborn et al., 2011)(Isaacs, Magallanes, Rebentrost, and Wight, 2009). UHPFRC is nearly impermeable hence, corrosion of embedded steel, from chemical ingress due to cracking and deterioration from freeze-thaw can be kept minimum and the stress-strain values are far more linear than any other type of material in its class, which allows it to gain an upper hand over normal-strength concrete (NSC) and high-performance concrete (HPC)(Ahlborn et al., 2011). On a recent account basis it has been stated that this material exhibits enhanced resilience against shock and impact loads, which has been established through laboratory and field measurements. Recent ballistic tests showed that 100 mm (4 in.) thick UHPFRC panels behave similar to a 190 mm (7.48 in.) of concrete block masonry (Isaacs et al., 2009). Since its first appearance in the market, this material has been used in numerous structural applications such as bridges, pavements, and architectural structures (Hassan et al., 2012). UHPFRC has great potential to be used in the bridge market in the United States and its greatest significance lies in the improved durability of concrete structures that leads to lower bridge repair costs and less downtime to repair.

Table 2.2 Mixture proportion design of UHPFRC
(Voo and Foster 2010)

Ingredients	Mass Kg/m³ (lbs/ft³)
UHPC premix	2100 (131.098)
Superplasticizer	40 (2.50)
Steel Fiber	157 (9.80)
Free Water	144 (8.99)
3% Moisture	30 (1.87)
Targeted W/B Ratio	0.15
Total Air Void	< 4%

Table 2.3 UHPC premix content
(Trinh, Bui Phuong 2012)

Materials	Properties		
	Density g/cm³ (lbs/in³)	Mean Particle Size (μm)	Chemical Composition SiO₂ (%)
Sand [S]	2.6 (0.09)	630	-
Cement PC 50 [C]	3.1 (0.11)	100	-
Silica Fume [SF]	2.2 (0.08)	8	> 95
Crushed Quartz [Q]	2.7 (0.01)	14	98.6

2.5 Physical and Chemical properties of UHPFRC

Table 2.4 displays the various characteristics strength values of UHPFRC which have been found by experiments referring to the relevant standards and codes.

Table 2.4 Characteristic strength of UHPFRC
(Voo and Foster 2010)

Sr. Nos.	Characteristics Properties	Units	Codes/Standards	Characteristics Values
1	Specific Density	kg/m ³ (lbs/ft ³)	BS1881:Part 114-1983	2350 – 2450 (146.71-152.94)
2	Cylinder Compressive Strength	Mpa (ksi)	AS1012.9-1999	120 – 160 (17.40 - 23.21)
3	Cube Compressive Strength	Mpa (ksi)	BS6319: Part 2-1983	130 – 170 (18.85 - 24.66)
4	Creep Coefficient at 28 days	-	ASTM C512	0.2 – 0.5
5	Post Cured Shrinkage	μ□	AS1012.16-1996	< 100
6	Modulus of Elasticity	GPa (ksi)	BS1881:Part 121-1983	40 – 50 (5801.51 - 7251.87)
7	Poisson's Ratio	-	BS1881:Part 121-1983	0.18 – 0.2
8	Split Cylinder Cracking Strength	Mpa (ksi)	ASTM C496	5 – 10 (0.725 - 1.45)
9	Split Cylinder Ultimate Strength	Mpa (ksi)	ASTM C496	10 – 18 (1.45 - 2.61)
10	Flexural Strength	N/mm ² (ksi)	ASTM C1018-1997	25.5 (3.70)
11	Modulus of Rupture	Mpa (ksi)	JCI-S-002-2003	18 – 35 (2.61 - 5.08)
12	Rapid Chloride Permeability	coulomb	ASTM C1202-2005	< 200
13	Chloride Diffusion Coefficient	mm ² /s	ASTM C1556-2004	0.05 – 0.1 x 10 ⁻⁶
14	Carbonation Depth	mm (in.)	BS:EN 14630-2003	< 0.1 (<0.004)
15	Abrasion Resistance	mm (in.)	ASTM C944-1999	< 0.03 (<0.001)
16	Water Absorption	%	BS1881:Part 122-1983	< 0.2

2.5.1 Compressive Strength

The most common test on any construction material is the compressive strength test of a cube or a cylindrical specimen. Hassan and his colleagues has followed the test configuration described in the BS 1881-121 and ASTM C469-94 for the determination of compressive stress–strain values and modulus of elasticity for both UHPFRC and UHPC (Hassan et al., 2012). However, there were some irregularities with the above guidelines and therefore some modifications were made to obtain the entire stress-strain range. Cylinders of size 50 mm (2 in.) diameter and 100 mm (4 in.) in length were tested by three different methods at 7, 14 and 28 days prior to casting. The strength of UHPFRC was in increasing order of 145 to 150 MPa (21 to 22 ksi) as per the days prior to casting with specimens behaving elastically up to approximately 90–95% of their compressive

strength, followed by strain hardening behavior (compression hardening) up to peak strength. While the strength of UHPC was in the range of 121 to 124 MPa (17.5 to 18 ksi) with the specimens behaving elastically until peak strength and then follow a sudden strain softening and lost their total strength at first cracking causing it to fail in an abrupt explosive manner. Trinh and Chanh prepared a cube specimen of size 100 mm (4 in.) for compression testing (Trinh and Van Chanh, 2012). The specimens were de-molded after 24 hours from casting and exposed to steam curing at 90°C for 2 days. After the end of curing period all the samples were tested for its compressive strength which was well above 140 MPa (20 ksi) for all samples and the maximum being 161 MPa (23.5 ksi). Corinaldesi and Moriconi prepared three prismatic specimens of size 40 x 40 x 160 mm (1.6 x 1.6 x 6.4 in.) to perform the compressive test with a variable of water to cement ratio and water to binder ratio (Corinaldesi and Moriconi, 2012). In terms of optimum workability and mechanical performance they found that the most recent type of acrylic based super-plasticizer (labeled 'spB') and a water to cement ratio of 0.24 were the best combination yielding compressive value of 156 MPa (22.6 ksi) along with flexural strength of 38 MPa (5.5 ksi). Kazemi and Lubell had prepared specimens with steel fibers (4% of volume) have a greater compressive strength of 18% and 6.7% at the age of 28 and 42 days as compared with specimens with no fibers. As the size of the specimens was doubled it was clearly evident from the data that the compressive strength has decreased by 13%. Also the cube specimens had higher compressive strength as compared to the cylindrical specimens of the respective size. For both compressive and tensile strengths, the mix with fibers 4% volume of steel fibers showed higher size effects than the plain

mixture with no fibers. The mechanical properties of UHPFRC were influenced by the specimen size. Decreases of 13%, 15% and 15% were noted for the compressive, flexural, and tension strengths respectively as the specimen sizes doubled (Kazemi and Lubell, 2011).

2.5.2 Tensile and Flexural Strength

UHPFRC can attain a strain hardening value of 0.15% with 2% of steel fiber (Volume). Hassan and his colleagues performed a direct tensile test on a two un-notched dog-bone specimens of overall length of 200 mm (8 in.) for UHPFRC and UHPC (Hassan et al., 2012). The obtained values were in the increasing order of 8 to 10 MPa (1.2 to 1.5 ksi) for UHPFRC while for UHPC they were in the range of 4 to 6 MPa (0.6 to 0.9 ksi). Trinh and Van Chanh performed a flexural strength test on a beam specimen of size 100 x 100 x 400 mm (4 x 4 x 16 in.) according to ASTM C1018 (Trinh and Van Chanh, 2012). All the samples yielded the flexural strength ranging between 14.6 to 19.5 MPa (2.1 to 2.8 ksi). Beams with and without steel fibers were tested for four point bending test which explicitly indicated that the use of steel fibers improves the flexural strength with an average increase of 31.2%. A 3-point bending test was also performed 100 mm x 100 mm x 400 mm (4 x 4 x 16 in.) specimens with fiber content between 1% to 5% in an increment of 1% in each sample (Su-Tae Kang, Lee, Yon-Dong Park, and Jin-Keun Kim, 2010). After failure, it is shown that one large crack exists, accompanying fibers thus due to the bridging mechanism of fibers, UHPFRC can provide superior performance especially under tension as compared to UHPC without fibers. It was observed that the flexural tensile strength of UHPFRC linearly increases as the fiber

volume ratio increases from 0% to 5%. According to Nematollahi and colleagues strain hardening can also be referred as tensile behavior of UHPFRC where the composite material continues to resist higher residual tensile strength after the concrete matrix has cracked. The post cracking strength of it is greater than the matrix cracking strength due to the use of very high strength micro steel fibers (Nematollahi et al., 2010). For the flexural test Kazemi and Lubell had prepared UHPFRC specimens with and without fibers which were tested according to ASTM C1609 (Kazemi and Lubell, 2011). Load versus deflection curves were plotted for all the specimens. The curves thus obtained showed clearly that the plain UHPFRC exhibited a very brittle behavior, whereas the mix with 4% of steel fiber showed a ductile behavior. For the specimens with steel fibers the flexural strength was increasing even after the formation of multiple micro-cracks whereas the plain mix showed a sudden failure after formation of first crack. The sole reason behind this extraordinary ability of UHPFRC is its microstructure. Microstructure of UHPFRC was observed using Scanning Electron Microscope (SEM) at 28 days. It revealed that the microstructure is very dense and has lower density of pores than ordinary concrete without silica fume. Portlandite ($\text{Ca}(\text{OH})_2$) crystals are reduced significantly due to the high content of silica fume in the mix which consumes the crystals during the process of hydration. The CH crystals are then converted to C-S-H gel (another chemical compound formed during hydration which is responsible for increasing the strength of concrete) and the transition zone was of very small thickness which indicates a good bond between the matrix and the aggregate surface as well as between the matrix and steel fibers.

2.5.3 Toughness test

Flexural toughness testing is the most common method of testing toughness of UHPFRC. The ASTM C 1018 standard has been used for toughness tests of fiber reinforced concrete according to which test results are made on the basis of dimensionless parameters of toughness indexes and residual strength factor. Nevertheless, there were many problems associated with ASTM C1018 and hence in the year 2005 the ASTM C 1018 standard was replaced with a new standard, i.e. ASTM C 1609 which yielded a more reliable value (Skazlic and Bjegovic, 2009). Marijan Skazlic and Dubravka Bjegovic performed ASTM C 1609 procedure for testing toughness of UHPFRC containing a large amount of fiber (more than 2% by volume) and exhibiting deflection hardening behavior. Flexural toughness and compressive strength tests were carried out on seven different UHPFRC mixtures having variable factors of steel fiber volume, its type, length and maximum aggregate grain size. Toughness was tested on 100 x 100 x 400 mm (4 x 4 x 16 in.) beams, while compressive strength was tested on 40 mm (1.6 in.) cubes. All the mixtures have compressive strength higher than 180 MPa (26.2 ksi) but the mean values of compressive strength exhibited by UHPFRC with 3% and 5% in volume of steel fibers were higher by 16% and by 17% respectively and by 16% and by 22% respectively compared with UHPFRC with 2% in volume of fibers. Load–deflection diagrams obtained from flexural toughness tests; deflection hardening after first-peak strength was noticed. Such deflection hardening was accompanied by multiple cracks and absorption of a large amount of energy (Skazlic and Bjegovic, 2009).

2.5.4 Creep and shrinkage

The creep behavior test was conducted by Lei Voo and Foster on UHPFRC over a period of 365 days, as per AS1012.16 (1996) on four pieces of 100 mm (4 in.) diameter by 200 mm (8 in.) high cylinders and were pre-loaded with at a compressive stress of 64 MPa (9.3 ksi). The test was conducted in an environmentally controlled room where the ambient temperature was maintained at 25°C and relative humidity at 50%. The creep coefficient of UHPFRC at 28 days was found to be 0.2 (Lei Voo and Foster, 2010). Trinh and Van Chanh measured shrinkage with and without steel fibers by casting a beam of size 100 x 100 x 400 mm (4 x 4 x 16 in.). The shrinkage was measured by using two LVDT attach to a metal rod 30 mm (1.2 in.) above the concrete surface on either supports. The sample with higher silica fume content indicates higher shrinkage ability and samples without steel fibers also had higher shrinkage ability (Trinh and Van Chanh, 2012).

2.5.5 Dynamic strength test

Many researches such as Suaris and Shah (1984), Banthia et. Al (1996), Gopalaratnam et al. 1986), Dancygier et al. (2007), Luo et al. (2000), Maalej et al. (2005) have concluded that UHPFRC exhibits better impact resistance than plain concrete after reviewing the explosive tests, projectile impact tests, drop weight impact tests and Charpy impact test. It exhibits less damaged area than plain concrete, it performs better than traditional reinforced concrete in high velocity projectile experiments, it provides increased shatter resistance with reduced scabbing, spalling, and fragmentation while exhibiting better energy absorption through distributed micro-cracking. The dynamic

behavior of brittle cementitious materials like UHPFRC is difficult to quantify since there aren't any standard approach. Gao X., Molyneaux T.C.K. and Patnaikuni, prepared four types of specimens for dynamic impact loading of UHPFRC; cylindrical 75 mm (3 in.) diameter x 150 mm (6 in.) length, cube 50 mm (2 in.), prismatic specimens 25 x 25 x 50 mm (1 x 1 x 2 in.) and small beams 280 x 70 x 70 mm (11 x 2.75 x 2.75 in.) for a three point flexural test. A customized drop hammer apparatus was developed for imparting impact loading on the test specimens. The results indicated that the flexural tensile strength Dynamic Increase Factor (DIF) of plain specimens is greater than UHPFRC specimens and also the compressive strength exhibits higher strain rate sensitivity in plain concrete compared with UHPFRC. The flexural tensile strength of UHPFRC specimens increases with the increase of strain rate from quasistatic to 4/sec as compared to 2/sec for plain concrete (Gao et al., 2008).

2.5.6 Durability

UHPFRC has a very dense matrix with very small and discontinuous pores which leads to extensively improved durability properties compared to Normal Strength Concrete (NSC) and High Performance Concrete (HPC). Furthermore steam-based curing increases the degree of hydration which results in enhancing of microstructure of the concrete body and reduces the permeability thus increasing the durability properties of UHPFRC. Durability is an indicator of concrete condition (health) which can be measured explicitly by performing the following test on the concrete samples.

2.5.6.1 Rapid chloride ion permeability

Trinh and Van Chanh carried out ASTM C1202 test for the Electrical indication of concrete's ability to resist chloride ion penetration frequently referred to as the rapid chloride ion penetrability test on UHPFRC samples having 50 mm (2 in.) thickness. The total charge passing through the samples were less than 100 coulombs which falls under the category of negligible limit of chloride ion permeability (Trinh and Van Chanh, 2012). Lei Voo and Foster performed chloride diffusion coefficient test on UHPFRC samples as per ASTM-C1556 (2004). The specimens were immersed in 4% chloride reagent for a period of 120 days before performing the test. The specimen without steel fiber has the highest value of $D_c = 0.136 \times 10^{-6} \text{ mm}^2/\text{s}$ and samples with 1% and 2% of steel fiber have average D_c values of 0.073×10^{-6} and $0.067 \times 10^{-6} \text{ mm}^2/\text{s}$ respectively. The values obtained speak for themselves as the sample without steel fibers has the highest value of chloride diffusion content. This value decreases as the volume of steel fibers in the mix increases (Lei Voo and Foster, 2010).

2.5.6.2 Porosity

The pores found in concrete mass can be distinguished into two types; gel pores and capillary pores. Gel pores occupy 28% of the total C-S-H volume and they are smaller than the molecule of water and hence it will check the flow of water. In addition to the above mentioned pores there are air voids, formed due to improper consolidation during casting or due to entrapped air. As the water to cement ratio increases the capillary porosity also increases resulting in a more permeable concrete. The interfacial transition zone is considered to be the most weakest and porous part of concrete. The following five

transport mechanisms can be studied to understand the ingress of chloride ions in concrete; hydrostatic advection, capillary suction, diffusion, thermal and electrical migration. In the presence of chloride ions if hydrostatic pressure exists on the concrete surface that can result in chloride penetration due to pressure gradient. Capillary suction is caused in a structure which is subjected to wet and dry cycles. If an under saturated concrete is exposed to chloride ions it will be drawn into the pore structure due to capillary suction. A process allowing aggressive ions to penetrate into concrete under the action of a concentration gradient is called diffusion. The rate of diffusion depends on the diffusion coefficient and the physical characteristics of the capillary pore structure (Ustabas, 2012). The chloride ions or molecules can travel faster in hot regions and portions of concrete which can be termed as thermal migration while the ions in the solution will attract towards the opposite electrode (anode) if the concrete specimen is subjected to electrical field (Shi et al., 2006).

2.5.6.3 Permeability test

There are three suggested mechanisms that explain the water repellent capability of UHPFRC such as self-sealing, autogenously healing and hydration of residual chloride. It was observed by Jean-Philippe Charron, Emmanuel Denarie and Eugen Bruhwiler in that the permeability and absorption increase steadily until a residual tensile deformation of 0.13% is reached in the material which results in seeping of water more distinctly. The above results were supported by the permeability test results indicating that the fluid flow in the UHPFRC rises with the increasing level of damage of the material. Since permeability is proportional to the cube of the crack width hence it takes

only one wide crack to make the material susceptible to transport the water to several micro-cracks (Charron, Denarie, and Bruhwiler, 2008). The liquid permeability of the UHPFRC was studied by J.P. Charron, E. Denarie and E. Bruhwiler with permeability cells constructed on the basis of Ludirdja et al. and Wang et al. work on a core specimen of UHPFRC without any steam treatment. Generally the liquid used for this kind of test is water but since it has been proven that UHPFRC has a large amount of un-hydrated cement which remains in the state of hydrating for many months after casting. Therefore water may not generate accurate data hence; glycol is used in this experiment instead of water. The UHPFRC specimens used for this test were obtained after performing the uniaxial tensile test where the inelastic deformation after unloading (ϵ) is around 0.13%. For the first 14 days, the curve is non-linear because the glycol is absorbed by the partially empty pores within the cracked specimen. After few days the permeability coefficients become nearly constant; meaning that a steady flow of glycol seeps through the specimens. The test results were clearly demonstrating that permeability increases with the level of deformation induced to the material. No significant influence of cracking can be observed on the equivalent permeability coefficient until deformation of unloading reaches to 0.13% which corresponds to a cumulated crack opening equal to 0.13 mm (0.005 in.) in UHPFRC sample while a crack opening of 0.05 mm (0.002 in.) is required to reach the same state of permeability in case of normal concrete. Since permeability is inversely proportional to surface roughness the value so obtained indicates that the path followed by water in the cracks is very complex and consequently long may be due to the presence of multi-branching cracks in the UHPFRC (Charron et

al., 2007). Hence UHPFRC is found to be far more superior material in restricting the flow of water through its body as compared with other types of concrete.

2.6 Ductal (grey premix)

Ductal is a new technology of ultra-high strength concrete which has constituted a breakthrough in concrete mix design. It has a very dense micro-structure and immensely high compressive strength which exceeds 200 MPa (29 ksi) (Paul Acker Lafarge, Paris France). It has the ability of keeping the hydration reaction of the mix within the prescribed limit and also contains particles of varying sizes to form a very dense matrix. It holds a self-filling function without changing the flow ability before and after the mixing of steel fibers. After steam curing, if applied, the cracking is controlled by the bridging effect of steel fibers even after the occurrence of initial cracking. It also has the ability of increasing its flexural resistance so that it can achieve flexural ductility even after the stresses reach the maximum value. Due to the high bond strength and high elastic modulus of steel fiber, it can be observed that the flexural resistance increases without a transient drop in stress or sharp increase in displacement after the initial cracking (Kida, Minakuchi, Abe, Sawano, and Katagiri, 2006). Passive reinforcements in structural elements can be avoided as a result of high ductility and compressive strength of Ductal (Paul Acker Lafarge, Paris France).

The basic principle used in Ductal is to minimize the defects like micro-cracks and pore spaces found in earlier type of concretes so that the resulting mixture is a dense matrix with increased load carrying capacity and enhanced durability properties. As per the above principle a concrete with maximum particle size of 0.1 mm (0.004 in.) was

proportioned to make a dense mixture with minimal voids and thus capable of yielding the following properties as shown in Table 2.5. Ductal research program was formulated on the basis of following principles:

- 1) Making the concrete mass more homogenous by elimination of coarse aggregate.
- 2) Having a proper gradation of particle size in the mix to increase the density of the concrete.
- 3) Application of post steam treatment to enhance its micro-structure.
- 4) Incorporate steel or PVA fibers in the matrix to add ductility to the concrete.
- 5) Maintaining mixing and casting procedure with a close proximity to existing practice.

Table 2.5 Characteristic properties of Ductal
(http://www.Ductal-lafarge.com/wps/portal/Ductal/6_5-Mechanical_performances)

Characteristics Properties	Ductal with Steel Fiber		Ductal with PVA Fiber
	TT	No TT	No TT
Density Kg/m ³ (lbs/ft ³)	2500 (156.07)		2350 (146.71)
Compressive strength Mpa (ksi)	150-200 (21.76-29)	150-180 (21.76-26.11)	100-140 (13-20.31)
Elastic limit in tension Mpa (ksi)	9-10 (1.31-1.45)	7-9 (1.01-1.31)	5-7 (0.725-1.01)
Flexural strength Mpa (ksi)	20-40 (2.90-5.80)	15-30 (2.18-4.35)	10-20 (1.45-2.90)
Young's Modulus Mpa (ksi)	45-55 (6.53-7.98)		35-45 (5.08-6.53)
Poisson's coefficient	0.2	0	0.2
Shrinkage mm/m	0.6-0.8		0.8-1
Creep	0.2-0.4	0.8-1	1-1.2
Thermal dilation (μm/m/°C)	12		12

TT = Thermal treatment

By incorporating the first three principles a concrete matrix was produced with a very high compressive strength but not very ductile. Hence the inclusion of fibers increases its ductility and tensile strength. After having done plenty of experiments by the Lafarge research department a final dosage and size of steel fibers was chosen; a content of 2% by volume and 13 mm (0.5 in.) in length and 0.2 mm (0.008 in.) in diameter of

steel fiber is the optimum combination to design Ductal for structural applications (Paul Acker Lafarge, Paris France).

2.7 Properties of Ductal with polyvinyl alcohol (PVA) fibers

The compressive strength and modulus of rupture of Ductal reinforced with polyvinyl alcohol fibers was calculated from a total of 67 batches; $152.3 \pm 6.3 \text{ MPa}$ ($22.1 \pm 0.9 \text{ ksi}$) (mean $\pm$ standard deviation) and $17.9 \pm 3.4 \text{ MPa}$ ($2.6 \pm 0.49 \text{ ksi}$), respectively (Adeeb et al., 2005). Shaheen and Shrive prepared three cylinders having a diameter of 2 inches (50 mm) and a height of 4 inches (100 mm) to test Ductal which was reinforced with 12 mm (0.5 in.) polyvinyl alcohol (PVA) fibers for determining its compressive strength. The average compressive strength of the three cylinders was calculated to be 136 MPa (20 ksi) and the Young's modulus was 53.3 GPa (7730 ksi). They also performed cyclic fatigue test on three un-notched beams having dimensions as 40 mm x 40 mm x 160 mm (1.6 x 1.6 x 6.3 in.). And it was observed that no failure had occurred by the complete separation of cracked surfaces for any of the specimens tested in fatigue no specimen broke into two pieces. Higher fatigue life was achieved due to the presence of PVA fibers which were bridging across the cracks. The fibers in the mix were experiencing different behavior: some of them were stretched below their elastic limit, some got partially pulled out, and some underwent plastic deformation while some maintained their geometry with no signs of loading. Specimens which were subjected to low and medium fatigue cycles caused their fibers to undergo splitting action but none of the specimens actually broke even after applying six million cycles hence, it can be concluded that these fibers can provide sufficient tensile strength. During the fatigue cycle the number of cycles was

recorded along with the associated crack length formed at that point of time. The crack length was increasing rapidly in the beginning but slowed down as the test progressed mainly because more and more fibers were brought into bridging action. The concrete tested showed no signs of corrosion due to the use of organic fibers but in the case of steel fibers the problem of corrosion can be encountered if a crack is exposed to environment otherwise the chances of corrosion are slim because Ductal mix provides very dense and low porosity matrix (Shaheen and Shrive, 2007).

2.8 Properties of Ductal with steel fibers

Years of research and experiments were conducted and recorded to come up with fixed characteristics values which can be applied by the designers and engineers in the field of designing and construction of realistic structures.

2.8.1 Compressive strength test

Theresa (Tess) M. Ahlborn, Devin K. Harris, Donald L. Misson, and Erron J. Peuse performed the compression test following the procedure of ASTM C39 as the baseline for the compression testing of Ductal (BS 1000) specimen. Specimens of size 3 in. x 6 in. (76 x 152 mm) and a compression machine having a capacity of 250 kips (1110 kN) were used and the load was applied at a stress rate of 150 psi/sec. The specimens were separated out on the basis of curing regimes and age. In order to find the most effective time for curing of UHPFRC and the best method of curing four different types of curing regimes were considered; air cured, TT (48-h thermal steam treatment), DTT (delayed thermal steam treatment: 10-day delay before curing is applied) and DDTT

(double-delayed thermal steam treatment: 24-day delay before curing is applied). Cylinders with fibers and with no fibers were used so that a comparative comparison can be concluded. The specimen with fibers experienced shear failure as characterized in C39 with fiber pull out and fiber breakage while specimens with no fibers had failed and broken into small pieces with an explosive noise in the test. The test data clearly indicated that the compressive stress was independent of curing regimes, age and addition of fibers. The mean stress of all the specimens was found to be 30.1 ksi (208 MPa). Although the air-cured specimens gained strength over the 28-day duration, there was no difference observed in the analysis of compressive stress after thermal treatment was applied. This trait allows a precaster to cast several elements over a period of time and then thermally treat them simultaneously, providing more flexibility in the casting and curing sequence (Ahlborn, Harris, Misson, and Peuse, 2011). Also Tetsukazu Kida and his colleagues carried out the compression test on a specimen having a diameter of 100 mm (4 in.) and length of 200 mm (8 in.). The compressive strength recorded by him was 216.3 MPa (31.4 ksi) (Kida et al., 2006). Compressive strength and bending behavior of Ductal have been measured and plotted on a graph as shown in Figure 3.1. It clearly showed that it has an ultimate bending strength which is over twice its first stress cracks and more than ten times the ultimate stress of conventional concrete. Hence this provides the designer's to design structures without any secondary passive and shear reinforcements (Paul Acker Lafarge, Paris France). Ductal performances exceptionally well in compression by displaying the fact that its compressive strength is 4 to 8 times higher than conventional concrete as shown in Figure 2.9. It has a linear elastic

compressive behavior up to the point maximum stress while exhibiting no damage to the material.

Figure 2.9 Compression curve of Ductal
(http://www.Ductal-lafarge.com/wps/portal/Ductal/6_5-Mechanical_performances)

2.8.2 Tensile and flexural test

Tetsukazu and his colleagues performed experiments of bending test (JIS R5201) after applying steam curing and splitting tensile strength test (JIS A1113) to determine the physical properties of Ductal with steel fibers having diameter of 0.2 mm (0.008 in.) and length of 15 mm (0.6 in.). The results obtained from the above test yielded the flexural strength to be 25.5 MPa (3.7 ksi), the cracking strength was 7.3 MPa (1.1 ksi) and the tensile strength was 11.8 MPa (1.7 ksi) respectively. Graphs were plotted between stress and strain from all the above tests and the displacement measured at the mid-span of the beam (Kida et al., 2006). The data obtained from them confirmed the following facts about Ductal, that the ultimate compressive strain of Ductal is twice as

large of ordinary concrete thus indicating that it is quite deformable, the volumetric strain was linear up till ultimate state thus indicating that Ductal has no critical points (cracking along the boundary of coarse aggregate and mortar paste) in the mix. The flexural stress of Ductal increases even after initial cracking, this tendency is shown by metals after yielding thus this is because of the bridging effect of steel fibers which are capable of controlling further cracking and retaining strength. The purpose of fibers in the Ductal mix is to provide the material a ductile behavior during bending as shown in Figure 2.10. In a case of flexure when the material is loaded beyond the elastic limit, there will be micro cracks occurring in the material which are closed tightly by the fibers in the mix providing a ductile performance rather than a sudden or brittle failure.

Figure 2.10 Displacement curve of Ductal
http://www.Ductal-lafarge.com/wps/portal/Ductal/6_5-Mechanical_performances)

2.8.3 Dynamic testing

For performing a dynamic compression test, a device called as split Hopkinson pressure bar (SPHB) often referred to as Kolsky bar was used. It not only allows the

loading of specimens at different strain rates but also has the capacity of measuring the strain and stress of the specimen at various strain rates. It is already a known fact that UHPFRC exhibits increasing strength with increasing loading rates. But it has been found that Ductal exhibits higher strengths than UHPFRC data at the same strain rates, in fact much higher strain rates were obtained in this study. Dynamic recovery tests indicated that Ductal samples can retain their load carrying capacity when partially damaged by an initial loading pulse. All the specimens obtained from the test had developed a diagonal shear plane on their surface which was bridged because of the presence of steel fibers. The same sample had further developed shear planes and was fragmented into small pieces after the application of second loading. The stress-strain curve generated from the above tests clearly indicates that Ductal retains significant levels of residual strength and stiffness when loaded beyond its peak strength (Isaacs, Magallanes, Rebentrost, and Wight, 2009).

The same test equipment as in the above case was used for the tension test on unconfined Ductal specimens. Similar to normal strength concrete the strength of Ductal decreases rapidly after reaching its peak strength but residual tensile strength was observed. Cracks were localized and assumed to be appearing across the voids or internal defects. Crack openings were prevented by the placement of steel reinforcing fibers perpendicular to the crack. The cracks had occurred in both the normal and parallel direction of the applied tension loading. “In tension, the material exhibits a post peak softening branch commonly observed in normal strength concrete, but retains over one third of its peak strength in residual tension strength” (Isaacs et al., 2009).

2.8.4 Static and dynamic load test

Static and dynamic (running wheel) load test were carried out on beams of varying thickness of 100, 150 and 200 mm (4, 6 and 8 in.) made up of Ductal. For the static case relatively dispersed cracks at the soffit of beam were developed beneath the point of application of load, ultimately the specimen failed in bending. Even in the dynamic case the beam failed in bending after causing cracks at wider range as compared to the former case of the smallest height because of the fact that the radius of curvature is smaller and hence more bridging effect is produced. An increase in the flexural load carrying capacity was seen with the increase in cross-sectional height of the beam. If a comparison can be made between static and dynamic load crack patterns it was observed that more fine cracks developed in the dynamic than the static case. Also comparing the flexural load capacity of both the cases the data obtained was very close for both the cases. There was no drop in the flexural capacity in the dynamic case as it is observed in the ordinary concrete. Hence it was concluded that Ductal is an excellent structural material for members subjected to continuous loading (Kida et al., 2006).

2.8.5 Modulus of elasticity and Poisson's ratio

Cylindrical specimens of size 3 in. x 6 in. (75 x 150 mm) were used and similar curing regimes were applied as in the case of compression test to act as a variable quantity. The specimens were prepared as described in Kollmorgen and Ahlborn et al and ASTM C469 test procedure was followed. The results were found to be similar to the compressive strength test. The modulus of elasticity of the air cured specimen was less compared to the other three curing regimes but was increasing by age. Like the

compressive stress samples, the three curing regimes had the same modulus of elasticity mean of 8,150 ksi (56.2 GPa). With thermal curing, the modulus value was increased by only 3.8%, whereas the compressive stress increased by 25.9% in comparison with air curing on the basis of 28-day data. When the fibers were removed from the mix the modulus of elasticity decreased. The four curing regimes had no impact on Poisson's ratio, as all specimens, independent of age or curing regimes, had the same mean of 0.21. So, the compressive strength, modulus of elasticity and Poisson's ratio are completely independent of the time at which thermal curing is applied. This information is useful to the designers, contractors and precasters since there is flexibility of time in casting and curing of materials (Ahlborn et al., 2011).

2.8.6 Rapid chloride penetration test

For rapid chloride penetration testing, ASTM C1202, Standard Test Method for Electrical Indication of Concrete's ability to resist chloride ion penetration, was followed for both specimen preparation and testing. The specimens were 2 in. (50 mm) in depth and 4 in. (100 mm) in diameter and exposed to NaCl solution. Three types of curing regimes were tested for chloride test that is 7-day TT, 28-day air-cured, and 28-day TT. All the measured data were under the acceptable limits but a key observation was made that the amount of charge passed for the thermally treated specimens was statistically lower than the air cured specimens. It should be remembered that the ASTM C1202 test does not measure any specific type of ion movement and instead measures bulk flow of ions through the specimen (Ahlborn et al., 2011).

2.8.7 Shrinkage and creep

Both shrinkage and creep are the remarkable properties of Ductal. The creep testing has been done in the United States at the Federal Highway Administration (FHWA) Research Center in McLean, Virginia, USA. The creep coefficient of Ductal is 0.8 and after the heat treatment it drops down to 0.2. Since the water to cement ratio of Ductal is impressively low Ductal does not exhibit drying shrinkage as shown in Figure 2.11. An endogenous shrinkage is observed 300 to 400 $\mu\text{m/m}$ but it is completely eradicated after applying a heat treatment and hence Ductal does not experience any residual shrinkage.

Figure 2.11 Creep curve of Ductal

(http://www.Ductal-lafarge.com/wps/portal/Ductal/6_5-Mechanical_performances)

2.8.8 Freezing and thawing cyclic test

To study the effect of freezing and thawing on Ductal, fourteen specimens of 3 in. $\times$ 4 in. $\times$ 16 in. (75 x 100 x 400 mm) Ductal (BS 1000) beams were subjected to ASTM C666 (Procedure B), Standard Test Method for resistance of concrete to rapid Freezing

and Thawing. Only two curing regimes were used ambient air and 7-day Thermal Treatment (TT). Available facilities in the laboratories were used each beam was subjected to at least 300 cycles. The deterioration caused by F-T was observed mechanically through the monitoring of a specimen's relative dynamic modulus (RDM) after every 32 cycles. In addition to the eight beams tested in the F-T chamber, six beams were cycled in and out of a separate water bath at ambient temperature to compare their mass change and RDM values to those of the specimens undergoing F-T cycling. All the measured data were under the acceptable limits of ASTM C666 and showed an increased RDM values as testing continued. Similar results were obtained with W-D specimens. The noticeable point was that the increases for the air-cured F-T specimens were significantly higher than the increases for the thermally treated F-T specimens. All the specimens were hydrating instead of deteriorating to F-T cycles and the air-cured W-D specimens RDM increased with a similar trend to that of the air-cured F-T specimens. However, the increase of the TT specimens was small in comparison to that of the air-cured specimens. This difference can be primarily attributed to the greater amounts of unhydrated cement particles in the air-cured specimens that can become hydrated in the presence of water. Plenty of other studies evidently indicate that, immersion of UHPFRC in water can increase its compressive strength and RDM even if the specimens are subjected to harsh environment (Ahlborn et al., 2011).

CHAPTER THREE

EXPERIMENTAL PROGRAM

This Chapter explains the research program in a comprehensive manner by explaining the type of materials and ingredients used in the preparation of the specimens. The composition, design and type of specimens used in the experimental test with their mixing and preparation procedures. Finally it explains the type, duration and number of experiments that were conducted on the samples to achieve the objectives of the research.

3.1 Materials

Three types of materials were used in this research; Lafarge Ductal, Quikrete® Non-Shrink Precision Grout and high early concrete mix (precast concrete). Each of them is explained in detail with their characteristics properties explained in Chapter 2.

3.1.1 Ductal grey premix

Ductal a commercial product name of Ultra High Performance Concrete (UHPC) manufactured by Lafarge (North America) was used in this research. It was used in combination with polyvinyl alcohol (PVA) or steel fibers as two different mixtures so as to produce UHPFRC. In this thesis, these mix designs will be referred to as *Ductal PVA* when PVA fibers were used and *Ductal Steel* when steel fibers were used. The Ductal mix designs used CHRYSO Fluid Premia 150, a high range water reducer (super-plasticizer). The two mix designs are shown in Table 3.1.

Table 3.1 Ductal mix designs

Material Combination	Fiber dimension in. (mm) [Diameter x Length]	Premix: Water: Fiber: HRWR (Ratio)
Ductal with Steel fiber	0.008 x 0.5 (0.20 x 12.7)	50 : 2.96 : 3.554 : 0.683
Ductal with PVA fiber	0.007874 x 0.75 (0.2 x 19.05)	50 : 3.527 : 0.866 : 0.677

3.1.2 Quikrete® Non-Shrink Precision Grout

Quikrete® Non-Shrink Precision Grout is a cementitious grouting material with a high fluid consistency property which can attain a compressive strength of 6000 psi (41.37 MPa) in seven days and 8000 psi (55.16 MPa) in 28 days. It can be classified into three types of consistency; plastic, flowable and fluid that can be achieved by changing the amount of water to be added to each bag (50 lbs.) of Quikrete. In this research fluid consistency was selected in order to comply with the similar standards that are adopted in the actual field of work on the construction site so that the results obtained could be compared with the field results with minimum amount of variable parameters between the lab and field work. Quikrete® Non-Shrink Precision Grout was used in combination with PVA fibers in the mix design as shown in the Table 3.2;

Table 3.2 Mix design of Quikrete Grout

Material Combination	Fiber dimension in. (mm) [Diameter x Length]	Premix: Water: Fiber (Ratio)
Quikrete with PVA fibers	0.001496 x 0.375 (0.038 x 9.52)	50 : 12.517 : 0.75

Quikrete® Non-Shrink Precision Grout is a high strength, non-metallic, Portland cement based material with expansive additives designed for grouting all types of machinery, steel columns, bearing plates, pre-cast concrete, and anchoring applications. It

will meet the specifications for ASTM C 1107, Standard Specification for Packaged Dry, Hydraulic-Cement Grout (Non-shrink), and CRD 621. It is a factory mixed non-metallic blend of selected fine aggregates, hydraulic cement, plasticizers, and shrinkage compensating additives. The product is approximately 50% cementitious and 50% high quality sand. It will not bond to metal. The Quikrete® grout placed around the metal will expand and act more like a “locking” mechanism where the metal would be tightly encapsulated within the grout. All material specifications for Quikrete® Non-Shrink Precision Grout can be found in Appendix A.

3.1.3 Precast concrete (High early concrete mix)

Precast concrete was procured from Metromont (Greenville, SC) for all the specimens prepared in this research. Mix design for per cubic feet of concrete used in the preparation of samples is shown in Table 3.3. For the mixing of the ingredients crushed aggregates and fine sand are mixed with 20% of batching water thoroughly. Then cement is added along with the rest of the batching water. HRWR is used and added at the end after cement. Total mix time of the ingredients is about 3 minutes. The average compressive strength of 28 days is about 7500 psi (51.71 MPa), Modulus of elasticity is about 4.02×10^6 psi (27716.93 MPa) and a unit weight of the concrete is 142 lbs./ft³ (2274.62 kg/m³). Sieve analysis and aggregate specifications used by Metromont in the preparation of concrete can be found in Appendix B.

Table 3.3 Mix design of concrete

Material	Quantity
Cement lbs. (kg)	650 (294.84)
Fly ash lbs. (kg)	100 (45.36)
Crushed stone lbs. (kg)	1750 (793.79)
Fine sand lbs. (kg)	1078 (488.97)
Water lbs. (kg)	280 (127)
W/C ratio	0.37

3.1.4 Mixing

Mixing UHPFRC (Ductal with steel or PVA fibers) requires special equipment and procedures to develop consistency in batching, casting, and curing in a timely fashion due to the low water cement ratio. Usually a mixer with high shear capacity, along with a vibratory table and a steam cure chamber capable of maintaining 100% humidity at 194°F (90°C), is required (Ahlborn et al., 2011) and recommended by the manufacturing agencies. On the construction field a high shear capacity mixer is available for mixing the materials and a needle vibrator or a tapping rod is used for applying compaction and the grouted concrete work is covered with a plastic sheet to prevent the loss of water from the mix in to the atmosphere instead of applying steam curing. Similarly, following procedure was applied in the preparation of specimens for this research a mixer with a high shear capacity was used to mix the materials and a tapping rod was used for applying the necessary compaction and in the end the grouted part was covered with plastic sheeting. Dry material (Ductal or Quikrete premix) were mixed till a homogenous mixture was built up; then in place of water ice (equivalent quantity) was used in order to keep the temperature of the mix within the prescribed limit since the mixing was carried

out in the hot summer day having an approximate temperature of 100°F. And then super-plasticizer was added and the mixture was mixed till a flowing and homogenous concrete was formed; next (Steel or PVA) fibers were added slowly and UHPFRC mixture was mixed till the fibers were well dispersed.

3.2 Specimens

There are two types of specimens used in this research; full-size specimen of a section of bridge deck with shear key (referred to as large samples) and modified ASTM G 109 samples. Step by step preparation of each type of samples has been explained in detail in the following verses.

3.2.1 Modified ASTM G 109 samples

There were five different types of modified ASTM G 109 samples with three of each kind so a total of 15 such samples were casted. Out of total six samples were casted using two different materials as shown in Table 3.4. A modified ASTM G 109 [42] mold was created for these samples. Instead of creating samples according to the standard and building a Plexiglas reservoir on the top, the reservoir was cast inside the sample. This was achieved by increasing the height of the mold such that it was equivalent to the sample height plus the height of the reservoir. A block of wood was then cut to the dimensions equal to that of the reservoir, and attached to the bottom of the mold with the help of nails (the samples were placed upside-down). The mold was prepared from plywood having a thickness of approximately quarter inch. Three holes were drilled on the opposite sides of each mold to accommodate the rebar and a wooden block of size 6 x

3 x 3 in. (1520 x 75 x 75 mm) was screwed in the middle of the mold to create a cavity for reservoir as shown in the Figure 3.1.

Table 3.4 G 109 sample details

Material	Date of casting precast concrete	Date of casting ductal	Nos. of sample	Starting date of corrosion test
Precast concrete	6/12/2012	-	3	7/11/2012
Ductal with PVA fiber	-	7/26/2012	3	8/28/2012
Ductal with Steel fiber	-	7/26/2012	3	8/28/2012
Precast concrete + Ductal with PVA fiber	6/12/2012	7/26/2012	3	8/28/2012
Precast concrete + Ductal with Steel fiber	6/12/2012	7/26/2012	3	8/28/2012

Figure 3.1 G109 mold

The #4 deformed rebar's which were used in G109 were slightly rusted as found on the construction site shown in Figure 3.2. One end of the rebar was drilled and tapped, allowing for the insertion of threaded steel rod, on which wires could be connected to the sample which are necessary for the corrosion testing of the embedded rebar. All the rebars were then power wire brushed for removing the rust and stains before epoxy

coating both the ends of the rebar as shown in Figure 3.3. Two coats of epoxy resin was applied on each end of the rebar in such way that an 8 in. (200 mm) middle portion of the rebar was kept uncoated. The reason for applying epoxy coats to the ends of the rebar was to protect the protruding section of the rebar from the either ends of the mold specimen from galvanic corrosion due to the effect of differential environment. The eight-inch section in the middle allowed the rebar, directly under the reservoir to corrode as shown in the Figure 3.4.

Figure 3.2 Rebar used in G 109 samples

Figure 3.3 Condition of rebar after wire brushing

Figure 3.4 Arrangement of rebar

Once the rebars were placed inside the molds concrete was filled in thirds. The concrete was tamped with a rod to ensure proper consolidation around the reservoir on the bottom of the mold and around the rebar. It was simple for nine of these specimens since they were filled in by one material alone but for the remaining six specimens were to be filled with precast concrete in one half and with Ductal in the other one half. A partition wall was made by using foam half inch thick and it was inserted roughly at the middle of the section. The molds were first filled in by precast concrete as shown in the Figure 3.5. Then the other half was filled in after a period of about six weeks with the required material and covered with plastic sheeting as shown in Figure 3.6. The entire purpose of filling one sample with two different materials was to create or replicate the interface of shear key with the precast concrete slab in the actual precast concrete adjacent box girder bridge construction. All necessary measures were taken to follow the

same procedure as adopted on the actual construction site so that the results procured from the tests can be applied to the actual structure.

Figure 3.5 G109 samples partially filled with concrete

Figure 3.6 G109 samples completely filled

The plastic sheeting was removed and the samples were demolded after three days from casting. Figure 3.7 and shows the image of demolded sample made of one material and two materials respectively. Water proofing compound was applied on all the samples throughout its surface area in two coats as shown in Figure 3.8. Only leaving the bottom of the samples part and the top surface of the reservoir so that corrosive agents like water, chloride ions and air can penetrate the samples from bottom and top surface only as it occurs in the real structures. Then after a period of 28 days from casting, all the samples were exposed to 3% NaCl solution.

(a)

(b)

**Figure 3.7 (a) G109 sample made of one material
(b) G109 sample made of two materials**

Figure 3.8 After applying water proofing compound

3.2.2 Full-size specimen of a section of bridge deck (large samples)

They can be further sub-divided into cracked large samples and un-cracked large samples. The procedure for the preparation of both types of samples was same but the difference being that the cracked samples were subjected to static and dynamic type of loadings whereas the non-cracked samples had not been subjected to any loadings prior to the start of corrosion experiments.

Cracked large samples

These samples comprise of three parts; shear key is the connection between the two precast concrete pieces as shown in the Figure 3.9. Two different materials were used in the casting of one large sample. Shear key was made of Ductal with steel fibers or PVA fibers or Quikrete with PVA fibers and the two adjacent slabs were made of precast concrete and the whole section was reinforced with #4 deformed bars in two directions perpendicular to each other. A total of five large cracked samples were used in the experiments. Table 3.5 provides the information regarding these cracked samples. They are differentiated on the basis of shear key materials and type of loadings and each one of them was exposed to NaCl solution and rest all other parameters being more or less same.

Figure 3.9 Elevation of large sample

Table 3.5 Large sample details

Sr. Nos.	Shear Key Material	Type of Loading	Designation
1	Quikrete with PVA fiber	Static	STA-01
2	Quikrete with PVA fiber	Fatigue	FAT-01
3	Ductal with Steel fiber	Static	STA-02
4	Ductal with Steel fiber	Fatigue	FAT-02
5	Ductal with PVA fiber	Static	STA-04

The length of one precast piece is 37 in. (940 mm) with its width being 40 in. (1020 mm) and having thickness (depth) of 8 in. (200 mm) while the shear key is 8 in. (200 mm) in width and 8 in. (203 mm) in depth cast between the entire widths of the two precast pieces therefore the total length of the large sample turns out to be 92 in. (2340 mm), with its width being 40 in. (1020 mm) and a depth of 8 in (200 mm). All reinforcing bars used throughout testing were #4, Grade 60 deformed rebar. #4 U rebar's as longitudinal reinforcement (bending steel) spaced at 8 in. center to center was aligned running parallel along the length of the precast slabs as shown Figure 3.10. Temperature and shrinkage #4 rebars were spaced at 12 in. center to center aligned parallel along the width of the precast slabs. Apart from bending and shrinkage reinforcement 2-#4 rebars were placed at the mid depth of the shear key running parallel to the width of the precast slabs and it was attached to the U bars protruding into the shear key from the either ends as shown in Figure 3.11. The longitudinal U rebar were arranged such that the reinforcement cover at the top was 2.5 in and 1 in. at the bottom.

Figure 3.10 Arrangement of steel reinforcement in large samples

Figure 3.11 Cross section of large sample

The precast slabs were cast at Metromont located in Greenville, SC and brought to the lab for joining the slabs by casting a shear key between them. Casting dates of slabs and shear keys are mentioned in the Table 3.6. The reinforcement bars used in the research were slightly rusted as found on a typical construction site. A precision control

protocol was followed in the preparation and casting of the samples throughout the research so that it can match with equivalent construction of the actual bridge in the field. Weeks before the casting of the shear key the interfaces of the precast slabs were sand blasted and then cleaned with air and water pressure. Couple of hours before the casting of the shear key the interfaces of slabs was rinsed with water using a water sprayer and care was taken that it remains wet until the casting of shear key. In case of casting Quikrete with PVA fibers the mixing was carried out in a conventional manner which lasted about 5 minutes. In case of Ductal with steel or PVA fibers a high shear energy mixer was used since mixing UHPFRC requires higher energy input to mix its ingredients to a level of achieving the desired flow ability of Ductal as shown in Figure 3.12. After filling the shear key it was covered with plastic sheeting for three days to prevent the loss of water content from the wet mix in to the atmosphere as shown in Figure 3.13.

Table 3.6 Dates of casting large samples (cracked)

Sample	Date of casting		Starting Date of corrosion test
	Slab	Shear Key	
STA - 01	1 25 2012	1 31 2012	7 13 2012
FAT - 01	1 25 2012	1 31 2012	7 13 2012
STA - 02	4 11 2012	4 27 2012	7 13 2012
FAT - 02	4 11 2012	4 27 2012	8 11 2012
STA - 04	3 29 2012	7 26 2012	8 29 2012

Figure 3.12 Ductal mixture

Figure 3.13 Curing of shear key

After casting the joint in the large samples it was cured at room temperature with minimum loss of moisture from its surface. The samples were then placed one at a time on a load actuator machine as shown in the Figure 3.14 to be subjected to either static or fatigue loadings. The static test lasted around 2 hours for the high moment test and 4.5 hours for the high shear test. The test is composed of several load levels. At each level, the specimen was loaded and unloaded twice until the final load level when the specimen was cracked at an extent beyond which it cannot be loaded any further. The fatigue test consists of applying 10 million cycles. The specimen was tested under a sine-wave load; the frequency applied was 5 Hz for the high moment test. Generally a fatigue test for one sample lasted 22 to 25 days, after which, 2 million cycles pond test was applied to check if there was any water seeping through the shear key interfaces. Then the specimen was loaded monotonically across its width until large deformation happened with a small load increment. The crack pattern and crack size formed on each sample was captured in photos and presented in Appendix C.

The cracked samples were then carefully removed from the actuator and placed on the wooden planks on the ground so that there is room for air to circulate beneath the specimen. With the help of diamond cutter saw a groove of size 60 in. (1500 mm) length and 30 in. (750 mm) in width was made on top of the sample (roughly in the middle). Then a plastic sheet material (plastic ABS) was inserted into the groove to act as a wall and sealed using silicon glue to be made as a pond which can be capable of holding water solution on top of the sample as shown in the Figure 3.15.

Figure 3.14 Loading machine

Figure 3.15 Large sample with a pond on top

Uncracked large samples

There were three uncracked large samples one type of each material as listed in the Table 3.7. The length and depth of these samples were same as the cracked large samples but the width of each sample being 10 in. (250 mm) as shown in the Figure 3.16. The width was reduced purposely to save time, cost and material since the area of interest and research was mainly concerned with the experimentation of the shear key and its interface with precast concrete on either side along the length of the specimen. Hence the width of the specimen was wide enough to encompass one #4 U-bar from either side of the slabs. There were no shrinkage and temperature reinforcements in these samples due to their small width. The preparation and casting operation was similar to the cracked samples and the difference being that these samples (uncracked) were not subjected to any kind of loadings. Hence after casting similar ponds were made on top of the samples to hold water solution.

Figure 3.16 Large samples (uncracked)

Table 3.7 Dates of casting large sample (uncracked)

Sample	Date of casting		Starting date of corrosion test
	Slab	Shear Key	
Ductal with Steel fiber	6/12/2012	7/26/2012	8/25/2012
Ductal with PVA fiber	6/12/2012	7/26/2012	8/25/2012
Quikrete with PVA fiber	6/12/2012	7/30/2012	8/25/2012

3.3 Experimental test setup

All the specimens were subjected to 3% NaCl solution in cycle of wet and dry periods each lasting for two weeks in order to accelerate the rate of corrosion of the embedded rebar. All the specimens were kept in the same laboratory at the same room temperature and relative humidity. There were three different types of experimental testing conducted on the large samples and the modified G109 samples: Half-cell Potential, Linear Polarization Resistance and Cyclic Polarization. All the experiments and their test setups are explained in detail in the following chapter.

3.3.1 Half-cell potential test (HCP)

Half-cell potential readings indicate the probability of corrosion that is taking place on the embedded rebar. Copper/copper sulfate (Cu/CuSO_4) reference electrode was used to measure the corrosion potential on the large samples while saturated calomel reference electrode was used on the G109 samples as shown in Figure 3.17. The reference electrode was placed directly on top of the concrete surface in case of wet measurements but in case of dry measurements a wet towel was placed between the reference electrode and concrete surface. A high impedance voltmeter was used to

measure the potential. Potential readings were taken at a distance of 6 in. from one point to the other marked on the surface of the samples in both vertical and horizontal directions as shown in the Figure 3.18. Two sets of readings were taken from one sample in each measurement cycle of wet and dry periods which were designated as left side when the voltmeter was connected to the rebar located on the left side and it was called right side when it was connected to the rebar on the right side as shown in the Figure 3.19. Measurements were taken regularly on each point of every sample after every two weeks in wet as well as dry cycles.

Figure 3.17 Copper/copper sulfate electrode and Saturated calomel electrode

Figure 3.18 Marking on large sample

Figure 3.19 Different sides of measurement

Same procedure was followed with the potential measurement of the G109 samples by using saturated calomel reference electrode. Potential readings of wet conditions were obtained instantly by placing the electrode on the surface of the concrete but 3 to 4 minutes (sometimes more time) were taken by the voltmeter to spit out a stable value for readings in dry conditions. In dry condition, a wet towel was placed between the reference electrode and the surface of the concrete.

3.3.2 Linear Polarization test setup

This test was conducted on points which had highest and lowest corrosion potential indicated by the half-cell potential test. SP -200 potentiostat was used to measure the polarization resistance (R_p) by using saturated calomel as reference electrode and a stainless steel shim as counter electrode. The shim was 5 in. (127 mm) long and 0.04 in. (1 mm) thick. Figure 3.20 shows the test setup used for all the electrochemical measurements, other than half-cell potential. There were three main wire connections to the equipment; to working electrode (corroding rebar), to counter electrode (steel shim) and reference electrode and the equipment was connected to the computer for measuring and recording the graphs by installing the EC-lab software. The reference electrode was fitted in a small piece of foam for balancing it to stand vertically on the surface. The steel shim was placed right below the electrode but not touching it. Both of these were placed on a wet cloth in case of dry condition measurement. Both wet and dry measurements were taken on the same selected points. The duration of the test on one point was about 2 minutes with a scan rate of 0.166 mV/s. Ohmic resistance was automatically compensated by the potentiostat.

Figure 3.20 Arrangement of LPR measurements

3.3.3 Visual observation

Visual inspection was performed to validate the results from all electrochemical measurements. Three cores were taken from each large sample at a point of maximum and minimum corrosion as indicted by the results of various corrosive measurement tests. The points of coring were selected such that one would lie in the center of the shear key, one at the interface of shear key and precast slab and one would lie in the precast slabs. Such variant locations were selected so that comparative results can be generated after observing the condition of the rebars. A heavy duty coring machine Dymodrill of Milwaukee brand as shown in Figure 3.21 was employed for the job of obtaining cores having a diameter of 2.25 in. and 8 in. depth, since the cores were taken throughout the depth of the sample. Top and bottom rebars were captured in each core sample. Two of the G109 samples of precast concrete and Ductal with PVA fibers were broken into two halves with a help of chisel and a hammer to retrieve the embedded top rebar. The samples made of Ductal with steel fibers were impossible to break with a hammer. Hence the same coring machine was used to take out cores from the remaining G109 samples. Each core was dried using compressed air and broken cautiously using a hammer to retrieve the rebar and concrete imprints. All pieces (concrete and the rebars) labeled and stored carefully in sealed plastic bag in a refrigerator. This procedure minimized further atmospheric corrosion.

Figure 3.21 Core machine

CHAPTER FOUR

RESULTS AND DISCUSSION

This Chapter discusses the results obtained from the various experiments conducted on the test specimens. Results and discussions are explained in a chronological order of tests carried out on different types of samples made of different types of materials; starting with modified G109 samples, uncracked large samples and cracked large samples.

4.1 Modified ASTM G109 samples

Five different concrete mixtures (three samples of each mixture which are referred to as sample 1, 2 and 3) were used to prepare the modified G109 samples and tested by electrochemical techniques and results were confirmed by visual examination of the rebar. Results of each set of samples are presented here. Pictures of concrete imprints of the core and samples are shown in Appendix D.

4.1.1 Precast concrete

Half-cell Potential test results

The HCP readings of all the three samples of precast concrete in both wet and dry conditions are presented in Figures 4.1 and 4.2 respectively. As observed in both graphs, sample 2 exhibit slightly more negative potential values than other samples. However, this difference is very small and negligible. According to Table 2.1 there is 90% probability of no corrosion in any of the samples. Little variation in potential lines can be

seen in wet condition as compared with the dry condition which displays a wider variation throughout the course of measurements in dry condition.

Figure 4.1 HCP graph of precast concrete in wet cycle

Figure 4.2 HCP graph of precast concrete in dry cycle

Linear polarization resistance test results

LPR test was carried out on only sample 2 which was expected to show slightly higher probability of corrosion activity compare to the other samples based on the HCP results. LPR test data of sample 2 are presented in Table 4.1.

Table 4.1 LPR data of precast concrete sample 2

Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m²	Condition
42	0.01093	Dry
43	0.00764	Dry
46	0.00763	Dry

All the measurements were taken in dry condition. The potentio-dynamic measurement proceeded as described in Chapter 3. The scan rate in all the cases was adjusted at 0.166 mV/s. According to the LPR results the rebar is in the active state of corrosion.

Visual examination (core test)

A complete top rebar was recovered from sample 2 of precast concrete as shown in Figure 4.3. There were absolutely no traces of corrosion products or corrosion activity on both the top and bottom part of the rebar along its length. The rebar was in the passive state and hence the visual test confirms the HCP test results in both conditions whereas it disapproves the LPR test data. It can be concluded that precast concrete with no cracks is capable of protecting the rebar from corrosion at least for a period of one year from the intrusion of chloride ions subjected on its top surface.

Top part

Bottom part

Figure 4.3 Precast concrete rebar

4.1.2 Ductal with PVA fibers

Half-cell Potential test results

The HCP test results in wet and dry conditions of all the Ductal PVA samples are presented in Figures 4.4 and 4.5, respectively.

Figure 4.4 HCP graph of Ductal PVA in wet cycle

Figure 4.5 HCP graph of Ductal PVA in dry cycle

As it is clearly evident from both graphs, sample 2 shows the higher corrosion activity that the other Ductal PVA samples with a probability of corrosion of more than 90%. The potentials of all the three samples are more negative in the wet measurements in comparison with the dry measurements.

Linear polarization test results

LPR test was carried out on sample 2 which had the highest probability of reinforcement corrosion as indicated by the HCP test. Results are presented in Table 4.2.

Table 4.2 LPR data of Ductal PVA sample 2

Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m^2	Condition
35	0.002609	Dry
36	0.001981	Dry
37	0.002674	Dry
39	0.001529	Dry

LPR test were taken in dry condition and all of the data suggest that there is an active corrosion in the embedded rebar. Thus the LPR test results coincide with that of the HCP test results.

Visual examination (core test)

A single core of the top rebar right below the pond from the center of the sample was extracted. The steel bar obtained from the core is shown in the Figure 4.6. HCP and LPR test results are justified and confirmed with the observation of the rebar's top surface which was under severe corrosion attack and also the rebar was found to be damp since the pond was filled with solution before extracting the core from the sample. This is a clear indication of solution seeping through the body of sample to reach the surface area of top rebar. The chances of solution to penetrate the top rebar other than top direction is negligible due to the application of two coats of water proofing compounds applied on the complete surface area of the sample as discussed in the earlier chapter. The above reasoning can be justified since the bottom part of the same rebar had no corrosion products at all as shown in the Figure 4.6. It can be concluded that the chloride ions present in the solution in the ponding on top penetrated into the sample having no visible cracks and travel downwards to a distance of about one inch to reach the top surface of the rebar.

Figure 4.6 Ductal PVA rebar

4.1.3 Ductal with steel fibers

Half-cell Potential test results

HCP test results in both wet and dry conditions are presented in the Figures 4.7 and 4.8, respectively. The potentials of all the three samples were very close to each other in both wet and dry cycles with sample 3 slightly being more negative than the other samples. All samples show a probability of corrosion of more than 90%.

Figure 4.7 HCP graph of Ductal Steel in wet cycle

Figure 4.8 HCP graph of Ductal Steel in dry cycle

Linear polarization resistance test results

LPR test was carried out on sample 3 of Ductal steel which is supposed to be the most corrosive of them all as per the HCP test results. LPR test results are presented in Table 4.3. The LPR test results indicate that the embedded rebar is in passive state and hence there is no corrosive activity occurring in it.

Table 4.3 LPR data of Ductal Steel sample 3

Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m²	Condition
35	0.000765	Dry
36	0.000771	Dry
39	0.000765	Dry

Visual examination (core test)

Single core was obtained from the center of the sample exactly beneath the pond to examine the condition of the top rebar as shown in Figure 4.9. No active corrosion was observed on the rebars. The LPR test results concur with the physical condition of the rebar. However, HCP test results indicated that there was 90% probability of corrosion. This could be due to the corrosion of the steel fibers not the embedded steel bar. It seems that the corrosion of steel fibers does not endanger the structure since corrosion products of the fibers do not penetrate into the UHPFRC matrix due to its 20 times less permeability than conventional concrete. Steel fibers will also expand due to corrosion but due to its small size and the strength of the surrounding matrix (UHPFRC) it will produce no spall due to internal stresses (Nematollahi et al., 2010).

Top part

Bottom part

Figure 4.9 Ductal Steel rebar

4.1.4 Precast concrete + Ductal with PVA fibers

Three samples made of precast concrete + Ductal with PVA fibers which will be referred to as PC + DP.

Half-cell Potential test results

HCP test results were carried out on all three PC + DP samples in both wet and dry conditions which are presented in the Figures 4.10 and 4.11 respectively. Potentials of all the three samples in the wet condition show fairly stable behavior with time in comparison with the potentials in the dry condition. Sample 1 had a clearly more negative potential in the wet condition with a steady increasing slope at the end of the test and had a more negative potential for much part of the dry period too, indicating the probability of more than 90% of corrosion of the embedded rebar as per the Table 2.1.

Figure 4.10 HCP graph of PC + DP in wet cycle

Figure 4.11 HCP graph of PC + DP in dry cycle

Linear polarization resistance test results

LPR test was carried out on dry condition on sample 1 of PC + DP with the most negative HCP value. LPR test results are presented in Table 4.4.

Table 4.4 LPR data of PC + DP sample 1

Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m^2	Condition
35	0.00488	Dry
36	0.00481	Dry
39	0.00285	Dry

LPR test results indicate that there is an active corrosion activity in the embedded rebar of sample 1 which concurs with the HCP test results in both the conditions.

Visual examination (core test)

Complete rebar was extracted from the sample 1 as shown in Figure 4.12 for visual examination. The HCP and LPR test results were found to be correct after finding the rebar to be corroded in both materials (Precast concrete and Ductal PVA). It can be hypothesized that the solution penetrated through the joint of two materials and initiated the corrosion process. The rebar embedded on the Ductal PVA side was found to be damp just as the one found in the plain Ductal PVA sample 2. In addition, it seems that the corrosion products formed on the top part of the rebar were far more intense than they were formed on the bottom part of the same rebar just as in the plain Ductal PVA sample.

Figure 4.12 PC + DP rebar

4.1.5 Precast concrete + Ductal with Steel fibers

Samples made of precast concrete + Ductal with steel fibers are referred to as PC + DS.

Half-cell Potential test results

HCP test results were carried out on all three PC + DS samples in both wet and dry conditions which are presented in the Figures 4.13 and 4.14 respectively. Steady lines of potential can be observed for all the three samples in the wet period. In the dry condition similar behavior is observed, with the exception of more variation. Sample 2 can be considered to have slightly higher probability of corrosion compare to the other samples.

Figure 4.13 HCP graph of PC + DS in wet cycle

Figure 4.14 HCP graph of PC + DS in dry cycle

Linear polarization test results

LPR test was carried out on sample 2 of PC + DS and the results are presented in Table 4.5. Comparison between values of Table 4.5 and Figures 4.13 and 4.14, shows that the LPR test results coincide with those obtained from the HCP test.

Table 4.5 LPR data of PC + DS sample 2

Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m^2	Condition
35	0.002147	Dry
36	0.001951	Dry
39	0.001892	Dry

Visual examination (core test)

Three cores from each part of the sample were extracted; from Ductal Steel (A), from the interface (B) and from precast concrete (C). Rebars extracted from the cores are shown in Figure 4.15, 4.16 and 4.17. Visual examination of the rebars confirms both the LPR and HCP test results. There was corrosion on the top part of the rebar found at the interface but no corrosion at the bottom part. Hence the solution must have penetrated through the joint between the two materials; precast concrete and Ductal steel.

Top part

Bottom part

Figure 4.15 Sample A

Ductal Steel Side

Precast Concrete Side

Top part

Bottom part

Figure 4.16 Sample B

Top part

Bottom part

Figure 4.17 Sample C

4.2 Uncracked large samples

There were three un-cracked large samples made up of three different materials. Half-cell potential (HCP) test was carried out on all the samples in wet and dry conditions. Then linear polarization resistance (LPR) test was carried out on three points of each sample; the first being on top of the shear key, second at the interface (between the joint of precast concrete and the shear key) and third being on top of precast concrete a point farthest away from the shear key. Their design configuration and construction procedure was similar to the actual bridge deck construction undertaken on the actual site. Copper/copper sulfate reference electrode was used to measure the half-cell potentials of large samples in both wet and dry conditions. Measurements of wet and dry cycles were taken at a period of two weeks apart from each other at the scheduled dates throughout the course of research. The LPR test results of only three points will be considered which will be called as sample 1, 2 and 3 which is capable of capturing the conditions of embedded rebar. Three cores were extracted from the three points at the end of the test period which will be referred to as shear key (sample 1), interface (sample 2) and precast concrete (sample 3) respectively as shown in Figure 4.18 from each type of sample for the purpose of visual examination of the rebar. The pictures of concrete imprints of all the samples can be seen in Appendix D.

Figure 4.18 Position of cores on the large uncracked

4.2.1 Ductal PVA

The sample is composed of two precast concrete slabs joined together by a shear key made of Ductal with PVA fibers. There was no sign of leakage or seeping of chloride solution between the interfaces of two different materials as shown in Figure 4.19 at the bottom part of the sample.

Figure 4.19 Bottom part of Ductal PVA interface

Half-cell potential test results

HCP test results of Ductal PVA are for both wet and dry conditions are displayed in Figures 4.20 and 4.21 respectively.

Figure 4.20 HCP graph of Ductal PVA in wet cycle

The three samples in the wet cycle started at the same potential and then there was a steady decline in the negativity of the potential values. There is a sudden jump in the potential lines of sample 2 and 3 whereas the potential line of sample 1 continues its steady decline till the end of the test period. The potentials of sample 2 and 3 fall under the category of increased probability of corrosion as indicated by the Table 2.1.

Figure 4.21 HCP graph of Ductal PVA in dry cycle

Similarly in the dry cycle the potentials of all the three samples started with less negative values and after approximately 20 weeks a sudden change in the potentials of the samples in the negative zone is observed just like the one in wet cycle. Sample 3 has the probability of increased corrosion while sample 1 has a probability of no corrosion while sample 2 falls between the two.

Linear polarization resistance test results

LPR test was carried out on two samples 2 and 3 of Ductal PVA in dry condition which are shown in Table 4.6. According to the LPR test results, the rebar is actively corroding. The LPR test results are in agreement with the HCP test results of sample 2 and 3.

Table 4.6 LPR data of Ductal PVA for sample 2 and 3

Sample Nos.	Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m^2	Condition
2	36	0.0028176	Dry
2	37	0.0028956	Dry
2	40	0.0056168	Dry
3	36	0.0050031	Dry
3	37	0.0079261	Dry
3	40	0.0085899	Dry

Visual examination of rebar (core test)

As said earlier, three cores were extracted from different parts of the sample to examine the surface of rebar as shown in Figure 4.22, 4.23 and 4.24. Both the rebar at top and bottom were corroded in all the core samples. Hence the HCP and LPR test results are confirmed with the visual inspection of the top rebar of the sample 2 and 3. However, according to the HCP results sample 1 should show no sign of active corrosion. This could be one of the deficiencies of the HCP technique. Unfortunately, no LPR was performed on the shear key (sample 1). However, the visual observations are in agreement with the observation from G 109 samples which showed active corrosion in Ductal +PVA material.

Top part

Bottom part

Figure 4.22 Top rebar of shear key (sample 1)

Ductal PVA Side

Precast Concrete Side

Top part

Bottom part

Figure 4.23 Top rebar of sample 2

Top part

Bottom part

Figure 4.24 Top rebar of sample 3

4.2.2 Ductal Steel

This sample was composed of two precast concrete slabs joined together by a shear key made of Ductal with steel fibers. There were signs of leakage or it can be seepage of chloride solution through the joint observed at the bottom part of the sample between the interfaces of two different materials as shown in Figure 4.25. The steel fibers which were protruding out from the shear key were found to be corroded.

Figure 4.25 Bottom part of Ductal Steel interface

Half-cell potential test results

HCP test results of Ductal Steel are for both wet and dry conditions are displayed in Figures 4.26 and 4.27 respectively. As can be seen, sample 1 shows less negative potentials compare to the other points, indicating a negligible probability of corrosion. Potentials of sample 2 and 3 were almost similar throughout the wet measurement period being well above the region of more than 90% chances of corrosion. In the dry cycle sample 1 follows the same pattern as it did in the wet cycle. Samples 2 and 3 follow a

similar pattern throughout the dry measurement period with more fluctuations. Sample 3 shows slightly more negative potential value as compared to sample 2.

Figure 4.26 HCP graph of Ductal Steel in wet cycle

Figure 4.27 HCP graph of Ductal Steel in dry cycle

Linear polarization resistance test results

LPR test was carried out on two samples 2 and 3 of Ductal steel are shown in Table 4.7. The LPR measurements are in agreement with the HCP test results that there is an active corrosion process in the embedded rebar.

Table 4.7 LPR data of Ductal Steel for sample 2 and 3

Sample Nos.	Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m²	Condition
2	37	0.0042067	Dry
2	40	0.003369	Dry
3	37	0.01269	Dry
3	40	0.014568	Dry

Visual examination of rebar (core test)

Three cores were extracted from different parts of the sample to examine the surface of rebar as shown in Figure 4.28, 4.29 and 4.30. Both the rebar at top and bottom were corroded in all the core samples. Hence the HCP and LPR test results are confirmed with the visual inspection for sample 2 and 3. However, corrosion is observed on sample 1 with minimum probability according to the HCP values. Unfortunately, no LPR measurements were performed on the shear key (sample 1).

Top part

Bottom part

Figure 4.28 Top rebar of sample 1

Ductal Steel

Precast Concrete

Top part

Bottom part

Figure 4.29 Top rebar of sample 2

Top part

Bottom part

Figure 4.30 Top rebar of sample 3

4.2.3 Quikrete with PVA fibers

This sample was composed of two precast concrete slabs joined together by a shear key made of Quikrete with PVA fibers. Signs of leakage or seepage of chloride solution through the joint were observed at the bottom part of the sample between the interfaces of two different materials as shown in Figure 4.31. There were salt deposits on the sides and bottom interfaces at the joint which was noticed only in this sample.

Side view of the interface

Bottom view of the interface

Figure 4.31 Interface of Quikrete PVA

Half-cell potential test results

HCP test results of Quikrete with PVA fibers for both wet and dry conditions are displayed in Figure 4.32 and 4.33 respectively.

Figure 4.32 HCP graph of Quikrete PVA in wet cycle

Figure 4.33 HCP graph of Quikrete PVA in dry cycle

The potentials of all the three samples in the wet cycle start at the same potential with sample 1 having slightly more negative potential as compared to other samples. Samples 2 and 3 have almost the same potentials along the entire wet measurement period which has a steady downward fall in its negative potential values. Sample 1 clearly exhibits a higher probability of corrosion (more than 90%) after 25 weeks of exposure to slat solution. Similar pattern is also observed in the dry cycle.

Linear polarization resistance test results

LPR test was carried out on just sample 2 of Quikrete PVA in dry condition which are shown in Table 4.8. The measurements of LPR test in the dry condition are in agreement with the HCP test results for sample 1 and shows active corrosion.

Table 4.8 LPR data of Quikrete PVA for sample 2

Sample Nos.	Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m²	Condition
2	36	0.0060474	Dry
2	37	0.0070539	Dry
2	40	0.0073469	Dry

Visual examination of rebar (core test)

Three cores were extracted from different parts of the sample to examine the surface of rebar as shown in Figures 4.34, 4.35 and 4.36. Both the rebar at top and bottom were slightly corroded in all the samples. Hence the HCP and LPR test results in both the conditions were accurate for sample 1 with the visual inspection of the top rebar of the samples. Rebar at samples 2 and 3 were also found to be corroded which exactly opposite to the HCP test results. Again this shows one of the difficulties with the HCP method.

Top part

Bottom part

Figure 4.34 Top rebar of sample 1

Ductal Steel Side

Precast Concrete Side

Top part

Bottom part

Figure 4.35 Top rebar of sample 2

Top part

Bottom part

Figure 4.36 Top rebar of sample 3

4.3 Cracked large samples

There were five cracked large samples having their shear keys made up of three different materials; Ductal with PVA, steel fibers and Quikrete with PVA fibers. They were tested under two types of loading; static and dynamic. There were no visible cracks on the top surface (exposed surface) of any of the samples Half-cell potential (HCP) test was carried out on all the samples in wet and dry conditions. With the help of half-cell potential values potential contour maps of all the samples with left side and right side measurements for wet and dry readings as explained in Chapter 3 were plotted using Microsoft Excel which can be seen in Appendix E. Then linear polarization resistance (LPR) test was carried out on three points on most of the samples; the first being on top of the shear key, second at the interface (between the joint of precast concrete and the shear key) and third being in precast concrete which is the point farthest away from the

shear key. Three cores were extracted from each sample from the above three points at the end of the test period for the purpose of visual examination of the rebar which will be referred to as sample 1, sample 2 and sample 3 respectively as shown in Appendix C for each sample respectively. The pictures of concrete imprints of all the samples can be seen in Appendix D.

4.3.1 Ductal PVA Static (STA - 04)

The sample was composed of two precast concrete slabs joined together by a shear key made up of Ductal with PVA fibers. This sample was subjected to static type of loadings and the cracks so formed at the bottom surface of the sample can be seen in Appendix C. There were signs of seeping of chloride solution between the cracks at the bottom surface of the sample as shown in Figure 4.37. Traces of salt deposits can be seen near the cracks hence there is a possibility that the solution might have seeped from top and reached the bottom of the surface through the non-visible cracks existing on the exposed top surface. But then again there is also a probability that the solution must have over flowed from the pond while filling it and dripped along the sides by the virtue of surface tension of material to reach the bottom surface of the sample. The test results of only three points will be considered as stated above which will be called as sample 1, 2 and 3 which is capable of capturing the conditions of embedded rebar at three different locations on the sample for deriving suitable conclusions.

Figure 4.37 Bottom surface of Ductal PVA Static

Half-cell potential test results

Copper/copper sulfate reference electrode was used to measure the half-cell potentials of large samples in both wet and dry conditions. Measurements of wet and dry cycles were taken at a period of two weeks apart from each other at the scheduled dates throughout the course of research. HCP test results of Ductal with PVA fibers for both wet and dry conditions are displayed in Figures 4.38 and 4.39, respectively. The potential line of all the three samples in the wet cycle starts and follows the same potential values throughout the entire wet measurement period. They have steady pattern indicating the active corrosion on the rebar.

Figure 4.38 HCP graph of Ductal PVA in wet cycle

Figure 4.39 HCP graph of Ductal PVA in dry cycle

The three samples in the dry cycle start almost at the same potential and then follow the different paths all along. Sample 2 has the probability of corrosion of more than 90% while sample 3 has the least chances of corrosion and sample 1 has its potentials between samples 2 and 3 with an increase chance of corrosion.

After careful examination of potential contour maps in wet cycle it can be stated that a uniformity of corrosion potential is achieved with the progress of time along the entire exposed surface area of the sample irrespective of the crack pattern. Also the corrosion potential values of each point were same even after changing the working electrode rebar. However, potential contour maps in the dry cycle are formed in complement with the crack patterns. The corrosion potential of points located in the areas of excessive cracks is increasing with the progress of time. Hence dry measurements are capable of producing more realistic representing models of corrosion potential.

Linear polarization resistance test results

LPR test was carried out on all the three locations of Ductal PVA (STA - 04) in dry condition which are shown in Table 4.9. As per the LPR test results all the rebar's are in an active state of corrosion. Thus LPR test data conveys the same results as HCP tests data and potential contour maps with sample 3 rebar being less corrosive than sample 2 rebar.

Table 4.9 LPR data of Ductal PVA Static (STA - 04)

Sample Nos.	Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m^2	Condition
1	36	0.010934	Dry
1	37	0.010175	Dry
2	36	0.020431	Dry
2	37	0.023416	Dry
2	40	0.032873	Dry
3	36	0.028332	Dry
3	37	0.02922	Dry
3	40	0.03663	Dry

Visual examination of rebar (core test)

Cores were extracted from different parts of the sample to examine the surface of rebar as shown in Figures 4.40, 4.41 and 4.42. Both the rebar at top and bottom of samples 1 and 2 were corroded with the bottom rebar being more corroded and sample 3 rebar was slightly corroded in comparison with the above samples. Bottom part of the rebars were more corrosive than the top part mainly because of the crack openings found at the bottom surface of the sample which allowed sufficient supply of oxygen and moisture to the bottom part of the rebar as well as the evaporation of the salt water from the other slab sitting beneath this slab. The HCP and LPR test results in both the conditions were confirmed with the visual inspection of the top rebar of the samples with the dry measurements being more accurate.

Top part

Bottom part

Figure 4.40 Top rebar of sample 1

Ductal PVA Side

Precast Concrete Side

Top part

Bottom part

Figure 4.41 Top rebar of sample 2

Top part

Bottom part

Figure 4.42 Top rebar of sample 3

4.3.2 Ductal Steel Static (STA - 02)

The sample was composed of two precast concrete slabs joined together by a shear key made of Ductal with steel fibers. This sample was subjected to static type of loadings and the cracks so formed at the bottom surface of the sample can be seen in Appendix C. There were signs of seeping of chloride solution at the bottom part of the sample between the cracks. The bottom surface of the sample was completely damped.

Half-cell potential test results

HCP test results of Ductal with steel fibers for both wet and dry conditions are displayed in Figures 4.43 and 4.44 respectively. The potential lines in the wet cycle of samples 1 and 2 have the same potentials and follow the same path throughout the wet period. Since the potential line is below samples 1 and 2 hence it is the noblest of the

three. According to their potential values there is 90% probability of no corrosion on all locations. The potential contour maps reveal that there is a uniform corrosion potential in the beginning till the end of the wet measurement period.

Figure 4.43 HCP graph of Ductal Steel in wet cycle

Figure 4.44 HCP graph of Ductal Steel in dry cycle

The potential line of only sample 2 in the dry cycle, up to 20 weeks, has the probability of corrosion of more than 90% while the potential lines of the other two are well below the datum line of probability of no corrosion. At the end of the dry measurement period and upward climb of potential lines of samples 1 and 3 is observed just like in the wet condition. Indicating the initiation of corrosion activity after lapsing a period of 40 weeks. Unlike the wet potential the dry potential measurement is influenced by the crack pattern as seen in the dry cycle potential contour maps in Appendix E. Clearly it can be seen that higher corrosion potential (more negative) value is measured at the bottom of bigger size crack openings and vice-versa. There are no significant changes in the corrosion potential except for slight drop in the potential value at the end of the measurement period.

Linear polarization resistance test results

LPR test was carried out on all the three samples of Ductal Steel (STA - 02) in dry condition which are shown in Table 4.10. As per the LPR test results all the rebars are in an active state of corrosion. Thus LPR test data is in complete agreement with the results of HCP tests data in dry condition and wet condition only for sample 2. Only visual observation of the rebar can reveal the accuracy of the above two test.

Table 4.10 LPR test data for Ductal Steel Static (STA - 02)

Sample Nos.	Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m²	Condition
1	36	0.003716	Dry
1	37	0.003997	Dry
1	40	0.002155	Dry
2	36	0.010981	Dry
2	37	0.008589	Dry
2	40	0.012239	Dry
3	37	0.016869	Dry
3	40	0.016075	Dry

Visual examination of rebar (core test)

As said earlier three cores were extracted from different parts of the sample to examine the surface of rebar as shown in Figures 4.45, 4.46 and 4.47. Both the rebar at top and bottom of samples 1 and 2 were corroded with the bottom rebar being more corroded and sample 3 rebar had small traces of corrosion products. Bottom part of the rebars was more corroded than the top part mainly because of the crack openings which allowed sufficient supply of oxygen and salt water evaporation. Hence the LPR test results were confirmed with the visual inspection of the top rebar of the samples.

Top part

Bottom part

Figure 4.45 Top rebar of sample 1

Precast Concrete Side

Ductal Steel Side

Top part

Bottom part

Figure 4.46 Top rebar of sample 2

Top part

Bottom part

Figure 4.47 Top rebar of sample 3

4.3.3 Ductal Steel Fatigue (FAT - 02)

The sample was composed of two precast concrete slabs joined together by a shear key made of Ductal with steel fibers. This sample was subjected to fatigue type of loadings and the cracks so formed at the bottom surface of the sample can be seen in Appendix C. There were no signs of seeping of chloride solution at the bottom part of the sample between the cracks as shown in Figure 4.48.

Figure 4.48 Bottom surface of Ductal Steel (FAT - 02)

Half-cell potential test results

HCP test results of Ductal with steel fibers for both wet and dry conditions are displayed in Figures 4.49 and 4.50 respectively. Potential lines of all the three samples start and follow the same path throughout the wet measurement period having a probability of corrosion of more than 90%. The contour maps reveal that there was almost uniform corrosion potential throughout the wet period with a decline in their values at the end of the wet measurement period.

Figure 4.49 HCP graph of Ductal Steel in wet cycle

Figure 4.50 HCP graph of Ductal Steel in dry cycle

The potential lines of samples 1 and 2 follow the same pattern in the dry cycle with almost same potential values forming couple of summits and having a probability of

corrosion of more than 90%. Sample 3 potential line is below the other two sample lines being the nobler among the three samples with a slight probability of corrosion. The dry potential contour maps follow the pattern of cracks with highest potential (more negative) values being near the shear key where the major cracks are located.

Linear polarization resistance test results

Due to some technical difficulties LPR test was not performed on this sample.

Visual examination of rebar (core test)

As said earlier three cores were extracted from different parts of the sample to examine the surface of rebar as shown in Figures 4.51, 4.52 and 4.53. Both the rebar at top and bottom of sample 1 and 2 were corroded with the bottom rebar being more corroded than the top rebar and sample 3 rebar had no traces of corrosion products at all. Bottom parts of the rebars were more corrosive than the top part mainly because of the crack openings. Hence the HCP test results in dry condition were in complete agreement with the visual inspection of the top rebar of the samples.

Top part

Bottom part

Figure 4.51 Top rebar of sample 1

Ductal Steel Side Precast Concrete Side

Top part

Bottom part

Figure 4.52 Top rebar of sample 2

Top part

Bottom part

Figure 4.53 Top rebar of sample 3

4.3.4 Quikrete PVA Fatigue (FAT - 01)

The sample was composed of two precast concrete slabs joined together by a shear key made of Quikrete with PVA fibers. This sample was subjected to fatigue type of loadings and the cracks so formed at the bottom surface of the sample can be seen in Appendix C. There were no visible signs of seeping of chloride solution at the bottom surface of the sample between the cracks as shown in Figure 4.54.

Figure 4.54 Bottom surface of Quikrete PVA Fatigue sample

Half-cell potential test results

HCP test results of Ductal with steel fibers for both wet and dry conditions are displayed in Figures 4.55 and 4.56 respectively. The potential lines of all the three samples have same potential values in the wet cycle with a probability of corrosion of uncertainty since the lines are around -300 mV potentials. They have a steady path with no rapid change in their potential lines having a constant bandwidth of potential values throughout the wet period. Also the corrosion potential over the entire top surface is uniform throughout the wet measurement period as observed in the potential contour maps.

Figure 4.55 HCP graph of Quikrete PVA in wet cycle

Figure 4.56 HCP graph of Quikrete PVA in dry cycle

The potential lines of all the three samples have identical pattern with different potential values in the dry cycle with couple of summits and a steep valley. Sample 1 having the highest probability of corrosion, sample 3 with the least probability of

corrosion and sample 2 sitting between the two. The corrosion potentials in the dry cycle are found to be highest (more negative) below the crack openings and low (less negative) at places with no cracks. Hence their pattern is influenced with the presence and absence of cracks and their opening sizes as shown in the contour map in Appendix E.

Linear polarization resistance test results

LPR test was carried out on two samples of Quikrete PVA (FAT - 01) in dry condition which are shown in Table 4.11. As per the LPR test data the rebar located at samples 1 and 3 are in active state of corrosion.

Table 4.11 LPR test data of Quikrete PVA Fatigue (FAT - 01)

Sample Nos.	Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m²	Condition
1	37	0.0192789	Dry
1	40	0.0177446	Dry
3	36	0.0330851	Dry
3	37	0.0298151	Dry
3	40	0.0294724	Dry

Visual examination of rebar (core test)

Three cores were extracted from different parts of the sample to examine the surface of rebar as shown in Figures 4.57, 4.58 and 4.59. Both rebars at top and bottom of sample 2 were corroded with the bottom rebar being more corroded than the top rebar and bottom rebar of sample 1 was slightly corroded while the top rebar had corrosion products and similarly there were some traces of corrosion products on sample 3 at bottom surface of top rebar. Bottom parts of the rebars were more corroded than the top part mainly because of the crack openings. Hence the HCP and LPR test results were in agreement with the visual inspection of the top rebar of the samples.

Top part

Bottom part

Figure 4.57 Top rebar of sample 1

Quikrete Side

Precast Concrete Side

Top part

Bottom part

Figure 4.58 Top rebar of sample 2

Top part

Bottom part

Figure 4.59 Top rebar of sample 3

4.3.5 Quikrete PVA Static (STA - 01)

The sample was composed of two precast concrete slabs joined together by a shear key made of Quikrete with PVA fibers. This sample was subjected to static type of loadings and the cracks so formed at the bottom surface of the sample can be seen in Appendix C. There were no visible signs of seeping of chloride solution at the bottom surface of the sample between the cracks as shown in Figure 4.60.

Figure 4.60 Bottom surface of Quikrete PVA Static (STA - 01)

Half-cell potential test results

HCP test results of Ductal with steel fibers for both wet and dry conditions are displayed in Figures 4.61 and 4.62 respectively. The potential lines of samples 1 and 2 share the same potential values in the wet cycle from start to end. After a period of 20 weeks it suddenly rises up from its normal path (more negative) indicating a sudden rise in the corrosion activity with a probability of more than 90% of corrosion of rebar. The corrosion potential along the entire top surface is uniform from starting to the end of the wet period with an increase in its value (more negative) indicative of increasing corrosion activity. Sample 3 is nobler with less negative potentials along the entire wet period compare to the other samples

Figure 4.61 HCP graph of Quikrete PVA in wet cycle

Figure 4.62 HCP graph of Quikrete PVA in dry cycle

All the three samples in the dry cycle follow a similar pattern. Sample 1 has the highest probability of corrosion and sample 3 has the least probability of corrosion and sample 2 has its potential lines between the potential lines of samples 1 and 2. As can be seen from the potential contour map (Appendix E), the values of corrosion potentials are highest (more negative) in the areas of cracking and it is lowest (less negative) in the areas free from cracking which indicates that the corrosion potentials in the dry cycle are influenced by the crack patterns found at the bottom surface of the sample.

Linear polarization resistance test results

LPR test was carried out on one sample of Quikrete PVA (FAT - 01) in dry condition which is shown in Table 4.12. According to the LPR data the rebar in sample 1 is under active corrosion process. Thus LPR test data corresponds with the HCP test results.

Table 4.12 LPR data of Quikrete PVA Static (STA - 01)

Sample Nos.	Time (weeks after exposure)	$i_{\text{corrosion}}$ in A/m^2	Condition
1	36	0.044984	Dry
1	37	0.046833	Dry
1	40	0.05544	Dry

Visual examination of rebar (core test)

Three cores were extracted from different parts of the sample to examine the surface of rebar as shown in Figures 4.63, 4.64 and 4.65. Both the rebar at top and bottom of samples 1 and 2 had no corrosion instead the top and bottom rebar of sample 3 had

some corrosion deposits. Hence the HCP and LPR test results in both dry and wet condition were found to be false with the visual inspection of the rebar. This is the only sample in which either of the test results could not match with the visual observation of the rebar.

Top part

Bottom part

Figure 4.63 Top rebar of sample 1

Quikrete Side

Precast Concrete Side

Top part

Bottom part

Figure 4.64 Top rebar of sample 2

Top part

Bottom part

Figure 4.65 Top rebar of sample 3

4.4 Summary of results

A summary of results is presented in the following tables for easy comparison between different tests and materials:

Table 4.13 Modified ASTM G 109 Samples

Position of the core	Test		Materials				
			Ductal Steel	Ductal PVA	Precast Conc.	PC + DP	PC + DS
At interface	HCP	Wet	-	-	-	C	C
		Dry	-	-	-	C	C
	LPR	Dry	-	-	-	C	C
	Core		-	-	-	C	C
Precast concrete	HCP	Wet	-	-	NC	C	C
		Dry	-	-	NC	C	C
	LPR	Dry	-	-	C	C	C
	Core		-	-	NC	C	T
Shear key	HCP	Wet	C	C	-	C	C
		Dry	C	C	-	C	C
	LPR	Dry	NC	C	-	C	C
	Core		NC	C	-	C	C

C = Corrosion

NC = No corrosion

T = Traces of corrosion

Table 4.14 Uncracked Large Samples

Position of the core	Test		Materials		
			Ductal PVA	Ductal Steel	Quikrete
At interface	HCP	Wet	C	C	NC
		Dry	C	C	NC
	LPR	Dry	C	C	C
	Core		C	C	C
Precast concrete	HCP	Wet	C	C	NC
		Dry	C	C	NC
	LPR	Dry	C	C	-
	Core		C	C	C
Shear key	HCP	Wet	NC	C	C
		Dry	NC	C	C
	LPR	Dry	-	-	-
	Core		C	C	C

C = Corrosion

NC = No corrosion

Table 4.15 Cracked Large Samples

Position of the core	Test		Materials				
			Ductal PVA (S)	Ductal Steel (S)	Ductal Steel (F)	Quikrete (S)	Quikrete (F)
At interface	HCP	Wet	C	C	C	C	C
		Dry	C	C	C	C	C
	LPR	Dry	C	C	-	-	-
	Core		C	C	C	NC	T
Precast concrete	HCP	Wet	C	NC	C	C	C
		Dry	NC	NC	NC	NC	C
	LPR	Dry	C	C	-	-	C
	Core		T	T	NC	T	T
Shear key	HCP	Wet	C	NC	C	C	C
		Dry	C	NC	C	C	C
	LPR	Dry	C	C	-	C	C
	Core		C	C	C	NC	T

C = Corrosion

NC = No corrosion

T = Traces of corrosion

S = Static sample

F = Fatigue sample

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

The conclusions of the whole research has been drawn from the results obtained from the experimental tests. The necessary suggestions are mentioned for future works that might be needed to be carried on the same topic in the future.

5.1 Conclusions

- 1) Generally, the half-cell potential contour map is a useful method in representing the corrosion activity of the rebars. This method is capable to detect the locations of the cracks with higher corrosion activities. However, it was found that the actual half-cell potential value cannot be used directly in some cases to determine the accurate probability of corrosion. HCP readings seems to be more accurate during measurements in dry periods.
- 2) The values obtained from the LPR test are more reliable in determining the corrosion activities in all cases.
- 3) Precast concrete and Ductal with steel fibers are capable of protecting the rebar from corrosion in the absence of surface cracks.
- 4) The embedded rebar in Ductal with PVA fibers exhibited corrosion activities. This could be due to the rapid fluid (salt solution) transport within the PVA fibers.
- 5) Corrosion activity was observed at the interface of two materials, shear key and precast concrete, regardless of the materials used in shear key. This indicates that the bonding at the interface is not strong enough to prevent chloride penetration and consequent corrosion.

5.2 Recommendations for the future works

The results and conclusions which have been derived in this preliminary research is only a starting step for carrying out a much more extensive research in the suitability of these new materials with the corrosion aspect of the embedded rebar. The following recommendations are suggested to carry out a detail investigation of the corrosion of the rebar embedded in such materials:

- 1) LPR should be performed more frequently probably once a week.
- 2) After corrosion is observed to be prominent, other corrosion tests such as cyclic voltammetry, cyclic polarization and other electrochemical techniques should be carried out on the steel bars for more precise investigation of the corrosion process.
- 3) New mix designs with varying fiber contents and varying fiber sizes (different length and diameter) needs to be adopted for deriving a more comparative results and conclusions which can be compared in terms of fiber volume and fiber aspects in the mix.
- 4) Chloride measurement techniques have to be included in the experimental test program to calculate the bounded and un-bounded chlorides in the concrete mass which can unfold many complications related with the various measuring techniques.
- 5) A detail investigation is needed for finding the transport property of fluids through concrete body due to the presences of PVA fibers in the mix by making

use of micro-structure techniques like X-ray radiography and X-ray CT scanning to find the orientation of the fibers in the mix after casting the samples.

- 6) To carry out the electrical resistance measurement of Ductal with steel and PVA fibers in comparison with that precast concrete.
- 7) Numerical models can be prepared by collecting data from this thesis to represent in a more realistic and appealing fashion.
- 8) To present the research in a conference for the purpose of awareness.

APPENDICES

Appendix A

Properties for Quikrete® Non-Shrink Precision Grout

NON-SHRINK PRECISION GROUT

PRODUCT NO. 1585-00

PRODUCT DESCRIPTION

QUIKRETE® Non-Shrink Precision Grout is a high strength, non-metallic, Portland cement based material with expansive additives designed for grouting all types of machinery, steel columns, bearing plates, pre-cast concrete, and anchoring applications.

PRODUCT USE

Typical applications for QUIKRETE® Non-Shrink Precision Grout include grouting of:

- All types of machinery
- Steel columns
- Bearing plates
- Precast concrete
- Other anchoring conditions that require high in-service strength

The non-shrink characteristics of Non-Shrink Precision Grout make it stable and capable of handling high load transfers.

SIZES

- QUIKRETE® Non-Shrink Precision Grout – 50 LB (22.7 kg) bags

YIELD

- Each 50 lb (22.7 kg) bag of QUIKRETE® Non-Shrink Precision Grout will yield 0.45 cu ft (12.7 L) at flowable consistency.

TECHNICAL DATA

APPLICABLE STANDARDS

ASTM International

- ASTM C109/C109M Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)
- ASTM C827 Standard Test Method for Change in Height at Early Ages of Cylindrical Specimens of Cementitious Mixtures
- ASTM C939 Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method)
- ASTM C1090 Standard Test Method for Measuring Changes in Height of Cylindrical Specimens from Hydraulic-Cement Grout
- ASTM C1107 Standard Specification for Packaged Dry, Hydraulic-Cement Grout (Nonshrink)
- ASTM C1107 Standard Test Method for Flow of Hydraulic Cement Mortar
- ASTM E488 Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements

U.S. Army Corps of Engineers (USACE) - CRD 621

DIVISION 3

Non-Shrink Grouting
03 62 00

PHYSICAL/CHEMICAL PROPERTIES

QUIKRETE® Non-Shrink Precision Grout complies with all properties of ASTM C1107 and CRD 621 producing the results shown in Table 1.

TABLE 1 TYPICAL PHYSICAL PROPERTIES AT 73°F (23°C)

Compressive strength, ASTM C109 modified per ASTM C1107

Plastic consistency	
1 day	3000 psi (20.7 MPa)
3 days	9500 psi (65.5 MPa)
7 days	10,000 psi (68.9 MPa)
28 days	14,000 psi (96.5 MPa)

Height change, ASTM C1090

1, 3, 7 and 28 days

0 - 0.2%

Height change, ASTM C827

+ 0.6%

Flowable consistency

1 day	3000 psi (20.7 MPa)
3 days	9000 psi (62.1 MPa)
7 days	9500 psi (65.5 MPa)
28 days	12,500 psi (86.2 MPa)

Height change, ASTM C1090

1, 3, 7 and 28 days

0 - 0.2%

Height change, ASTM C827

+ 0.4%

Fluid consistency

1 day	2500 psi (17.2 MPa)
3 days	5000 psi (34.5 MPa)
7 days	6000 psi (41.4 MPa)
28 days	8000 psi (55.2 MPa)

Height change, ASTM C1090

1, 3, 7 and 28 days

0 - 0.2%

Height change, ASTM C827

+ 0.3%

Pull-out strength, ASTM E488†

35,000 psi (241 MPa)

[†] 1 1/4" (31 mm) bolts embedded 9" (225 mm) deep in 3" (75 mm) hole in 2000 psi (13.8 MPa) concrete.

INSTALLATION

SURFACE PREPARATION

Surfaces to receive the grout must be clean and free of any type of foreign matter, grease, paint, oil, dust or efflorescence. In some cases it may be necessary to roughen smooth surfaces or etch old ones with acid. The area should be flushed and soaked with clean water prior to grouting leaving no standing water. Place the grout quickly and continuously using light rodding to eliminate air bubbles.

MIXING

Add the minimum amount of water necessary to produce the desired flow characteristics as indicated in Table 2. Do not add more water than the amount needed to produce a 20-second flow per ASTM Test Method C 939. QUIKRETE® Non-Shrink Precision Grout should be mechanically mixed for a minimum of 5 minutes.

TABLE 2

APPROXIMATE WATER REQUIRED FOR 50 LB (22.7 KG) OF GROUT	
Plastic	1 gal (3.8 L)
Flowable	1 gal + 1 pt (4.3 L)
Fluid	1 gal + 3 pt (5.2 L)

WORKING TIME

When properly mixed to a fluid consistency QUIKRETE® Non-Shrink Precision Grout will comply with all portions of ASTM C1107 and CRD 621 and retain a fluid consistency for the maximum usable working times stated in Table 3.

TABLE 3

WORKING TIME

Temperature	Working time
50°F (10°C)	25 min
73°F (23°C)	25 min
90°F (32°C)	15 min

CURING

A damp cure of at least 3 days is necessary to control the non-shrink characteristics and maintain strength levels.

PRECAUTIONS

- Additions of cement or other materials will eliminate the designed product qualities
- Water quantities may be affected by temperature, mixing method and batch size
- QUIKRETE® Non-Shrink Precision Grout should not be re-tempered
- Grout temperature should be maintained from 50 - 90 degrees F (10 - 32 degrees C) to achieve specified results. Use cold water in hot weather or hot water in cold weather to achieve desired grout temperature.
- Do not pour grout if temperature is expected to go below 32 degrees F (0 degrees C) within a 12 hour period.
- Mix no more than can be used in 30 minutes

WARRANTY

The QUIKRETE® Companies warrant this product to be of merchantable quality when used or applied in accordance with the instructions herein. The product is not warranted as suitable for any purpose or use other than the general purpose for which it is intended. Liability under this warranty is limited to the replacement of its product (as purchased) found to be defective, or at the shipping companies' option, to refund the purchase price. In the event of a claim under this warranty, notice must be given to The QUIKRETE® Companies in writing. This limited warranty is issued and accepted in lieu of all other express warranties and expressly excludes liability for consequential damages.

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* Refer to www.quikrete.com for the most current technical data, MSDS, and guide specifications

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Shrink%20Precision%20Grout%201585-00.pdf

Appendix B

Aggregate Properties

Source: Metromont manufacturing plant (Greenville)

Appendix C

Crack patterns of large samples

1) Quikrete with PVA fibers (STA - 01)

Plan

Elevation

Crack sizes

2) Quikrete with PVA fibers (FAT - 01)

Plan

Elevation

Crack sizes

3) Ductal with steel fibers (STA - 02)

Elevation

Crack sizes

4) Ductal with steel fibers (FAT - 02)

5) Ductal with PVA fibers (STA - 04)

Plan

0.080"
C

0.060"
B

0.125"
A

JH

0.040"
D

Elevation

Crack sizes

Appendix D

Visual examination of the rebar (cores)

1) Modified ASTM G109 samples

i) Precast concrete (Sample 2)

Figure D.1 Bottom part of precast concrete

ii) Ductal PVA (Sample 2)

Top part

Bottom part

Figure D.2 Concrete imprints of Ductal PVA

iii) Ductal Steel (Sample 3)

Top part

Bottom part

D.3 Concrete imprints of Ductal Steel

iv) Precast concrete + Ductal PVA (Sample 1)

Top part

Bottom part

D.4 Concrete imprints of PC + DP

v) Precast concrete + Ductal Steel fibers (Sample 2)

Top part

Bottom part

Figure D.5 Concrete imprints of sample B

Figure D. 6 Top part of sample C

2) *Un-cracked large samples*

i) Ductal PVA

Top part

Bottom part

Figure D.7 Concrete imprints of sample 1

Figure D.8 Top part of concrete imprint of sample 2

Figure D.9 Top part of concrete imprint of sample 3

ii) Ductal Steel

Top part

Bottom part

Figure D.10 Concrete imprints of sample 1

Top part

Bottom part

Figure D.11 Concrete imprints of sample 2

Top part

Bottom part

Figure D.12 Concrete imprints of sample 3

iii) Quikrete PVA

Figure D.13 Concrete imprints of top part of sample 1

Top part

Bottom part

Figure D.14 Concrete imprints of sample 2

Top part

Bottom part

Figure D.15 Concrete imprints of sample 3

3) Cracked large samples

i) Ductal PVA Static (STA – 04)

Figure D.16 Bottom part of concrete imprint of sample 1

Top part

Bottom part

Figure D.17 Concrete imprint of sample 2

Figure D.18 Top part of concrete imprint of sample 3

ii) Ductal Steel Static (STA - 02)

Top part

Bottom part

Figure D.19 Concrete imprint of sample 2

iii) Ductal Steel Fatigue (FAT - 02)

Figure D.20 Bottom part of concrete imprint of sample 2

Figure D.21 Bottom part of concrete imprint of sample 3

iv) Quikrete PVA Static (STA - 01)

Top part

Bottom part

Figure D.22 Concrete imprint of sample 1

Figure D.23 Concrete imprint of top part of sample 2

Top part

Bottom part

Figure D.24 Concrete imprint of sample 3

v) Quikrete PVA Fatigue (FAT - 01)

Figure D.25 Concrete imprint of top part of sample 1

Figure D.26 Concrete imprint of top part of sample 2

Figure D.27 Concrete imprint of top part of sample 3

Appendix E

Potential contour maps of cracked large samples

1) Ductal PVA Static (STA - 04)

Wet cycle

First reading

Last reading

Figure E.1 Wet cycle contour maps

Dry cycle

Figure E.2 Dry cycle contour maps

2) Ductal Steel Static (STA - 02)

Wet cycle

Figure E.3 Wet cycle contour maps

Dry cycle

First reading

Last reading

Figure E.4 Dry cycle contour maps

3) Ductal Steel Fatigue (FTA - 02)

Wet cycle

Figure E.5 Wet cycle contour maps

Dry cycle

First reading

Last reading

Figure E.6 Dry cycle contour maps

4) Quikrete PVA Fatigue (FAT - 01)

Wet cycle

Figure E.7 Wet cycle contour maps

Dry cycle

Figure E.8 Dry cycle contour maps

5) Quikrete PVA Static (STA - 01)

Wet cycle

Figure E.9 Wet cycle contour maps

Dry cycle

Figure E.10 Dry cycle contour maps

References

- Abosrra, L., Ashour, A. F., & Youseffi, M. (2011). Corrosion of steel reinforcement in concrete of different compressive strengths. *Construction and Building Materials*, 25(10), 3915-3925. doi:10.1016/j.conbuildmat.2011.04.023
- Adeeb, S. M., Scholefield, B. W. J., Brown, T. G., Shrive, N. G., Kroman, J., Perry, V. H., & Tadros, G. (2005). Modelling, testing, and construction of the first Ductal® canopy in the world. *Canadian Journal of Civil Engineering*, 32(6), 1152-1165. doi:10.1139/l05-065
- Ahlborn, T. M., Harris, D. K., Misson, D. L., & Peuse, E. J. (2011). Characterization of strength and durability of ultra-high-performance concrete under variable curing conditions. *Transportation Research Record*, (2251), 68-75. doi:10.3141/2251-07
- Ahmad, S. (2003). Reinforcement corrosion in concrete structures, its monitoring and service life prediction - A review. *Cement and Concrete Composites*, 25(4-5), 459-471. doi:10.1016/S0958-9465(02)00086-0
- Al-Amoudi, O., & Maslehuddin, M. (1993). Effect of chloride and sulfate ions on reinforcement corrosion. *Cement and Concrete Research*, 23(1), 139-146. doi:10.1016/0008-8846(93)90144-X
- Angst, U. M., Elsener, B., Larsen, C. K., & Vennesland, O. (2011). Chloride induced reinforcement corrosion: Electrochemical monitoring of initiation stage and chloride

threshold values. *Corrosion Science*, 53(4), 1451-1464.

doi:10.1016/j.corsci.2011.01.025

Ann, K. Y., & Song, H. (2007). Chloride threshold level for corrosion of steel in concrete. *Corrosion Science*, 49(11), 4113-4133. doi:10.1016/j.corsci.2007.05.007

Arya, C., & Ofori-Darko, F. (1996). Influence of crack frequency on reinforcement corrosion in concrete. *Cement and Concrete Research*, 26(3), 345-353.

Aveldano, R. R., & Ortega, N. F. (2011). Characterization of concrete cracking due to corrosion of reinforcements in different environments. *Construction and Building Materials*, 25(2), 630-637. doi:10.1016/j.conbuildmat.2010.07.029

Barnett, S. J., Lataste, J., Parry, T., Millard, S. G., & Soutsos, M. N. (2010). Assessment of fibre orientation in ultra high performance fibre reinforced concrete and its effect on flexural strength. *Materials and Structures/Materiaux Et Constructions*, 43(7), 1009-1023. doi:10.1617/s11527-009-9562-3

Bhaskar, S., Gettu, R., Bharatkumar, B. H., & Neelamegam, M. (2011). Studies on chloride induced corrosion of reinforcement steel in cracked concrete. *Structural Durability & Health Monitoring*, 7(4), 231-51.

Broomfield, J. P., Davies, K., & Hladky, K. (2002). The use of permanent corrosion monitoring in new and existing reinforced concrete structures. *Cement and Concrete Composites*, 24(1), 27-34. doi:10.1016/S0958-9465(01)00024-5

- Charron, J. -, Denarie, E., & Bruhwiler, E. (2007). Permeability of ultra high performance fiber reinforced concretes (UHPFRC) under high stresses. *Materials and Structures/Materiaux Et Constructions*, 40(3), 269-277. doi:10.1617/s11527-006-9105-0
- Charron, J. -, Denarie, E., & Bruhwiler, E. (2008). Transport properties of water and glycol in an ultra high performance fiber reinforced concrete (UHPFRC) under high tensile deformation. *Cement and Concrete Research*, 38(5), 689-98.
doi:10.1016/j.cemconres.2007.12.006
- Chen, D., & Mahadevan, S. (2008). Chloride-induced reinforcement corrosion and concrete cracking simulation. *Cement and Concrete Composites*, 30(3), 227-38.
doi:10.1016/j.cemconcomp.2006.10.007
- Cho, S. -, Chung, L., & Roh, Y. (2005). Estimation of rebar corrosion rate in reinforced concrete structure. *Corrosion Reviews*, 23(4-6), 329-353.
- Corinaldesi, V., & Moriconi, G. (2012). Mechanical and thermal evaluation of ultra high performance fiber reinforced concretes for engineering applications. *Construction and Building Materials*, 26(1), 289-294. doi:10.1016/j.conbuildmat.2011.06.023
- Feliu, S., Gonzalez, J. A., & Andrade, C. (1996). Electrochemical methods for on-site determinations of corrosion rates of rebars. Paper presented at the *Proceedings of the 1994 Symposium on Techniques to Assess the Corrosion Activity of Steel Reinforced Concrete, December 7, 1994 - December 7, , 1276* 107-118.

- Gao, X., Molyneaux, T. C. K., & Patnaikuni, I. (2008). Dynamic impact testing of ultra high performance fibre reinforced concrete. Paper presented at the *20th Australasian Conference on the Mechanics of Structures and Materials, ACMSM20, December 2, 2008 - December 5*, 631-636.
- Gowers, K. R., & Millard, S. G. (1993). On-site linear polarization resistance mapping of reinforced concrete structures. *Corrosion Science*, 35(-8), 1593-1600.
- Gowers, K. R., & Millard, S. G. (1999). Measurement of concrete resistivity for assessment of corrosion severity of steel using wenner technique. *ACI Materials Journal*, 96(5), 536-541.
- Hanna, K. E., Morcous, G., & Tadros, M. K. (2009). Transverse post-tensioning design and detailing of precast, prestressed concrete adjacentbox-girder bridges. *PCI Journal*, 54(4), 160-174.
- Hassan, A. M. T., Jones, S. W., & Mahmud, G. H. (2012). Experimental test methods to determine the uniaxial tensile and compressive behaviour of ultra high performance fibre reinforced concrete(UHPFRC). *Construction and Building Materials*, 37, 874-882. doi:10.1016/j.conbuildmat.2012.04.030
- Hussain, S. E., Al-Gahtani, A., & Rasheeduzzafar. (1996). Chloride threshold for corrosion of reinforcement in concrete. *ACI Materials Journal*, 93(6), 534-538.

- Isaacs, J., Magallanes, J., Rebentrost, M., & Wight, G. (2009). Exploratory dynamic material characterizing tests on ultra-high performance fibre reinforced concrete. Paper presented at the *8th International Conference on Shock and Impact Loads on Structures, December 2, 2009 - December 4, 337-344*.
- Jaffer, S. J., & Hansson, C. M. (2008). The influence of cracks on chloride-induced corrosion of steel in ordinary portland cement and high performance concretes subjected to different loading conditions. *Corrosion Science*, 50(12), 3343-55. doi:10.1016/j.corsci.2008.09.018
- Kato, E., Kato, Y., & Uomoto, T. (2005). Development of simulation model of chloride ion transportation in cracked concrete. *Journal of Advanced Concrete Technology*, 3(1), 85-94.
- Kazemi, S., & Lubell, A. S. (2011). Size effects on the mechanical properties of UHPFRC. Paper presented at the *Annual Conference of the Canadian Society for Civil Engineering 2011, CSCE 2011, June 14, 2011 - June 17, 2011*, 2 1256-1265.
- Kida, T., Minakuchi, K., Abe, T., Sawano, T., & Katagiri, M. (2006). Experimental study on mechanical properties of ultra high strength fiber reinforced concrete beam. *Theoretical and Applied Mechanics Japan*, 55, 83-88.
- Kouril, M., Novak, P., & Bojko, M. (2006). Limitations of the linear polarization method to determine stainless steel corrosion rate in concrete environment. *Cement and Concrete Composites*, 28(3), 220-225. doi:10.1016/j.cemconcomp.2006.01.007

- Law, D. W., Millard, S. G., & Bungey, J. H. (2000). Linear polarization resistance measurements using a potentiostatically controlled guard ring. *NDT and E International*, 33(1), 15-21. doi:10.1016/S0963-8695(99)00015-8
- Lei Voo, Y., & Foster, S. J. (2010). Characteristics of ultra-high performance 'ductile' concrete and its impact on sustainable construction. *IES Journal Part A: Civil and Structural Engineering*, 3(3), 168-187. doi:10.1080/19373260.2010.492588
- Mangat, P. S., & Molloy, B. T. (1991). Influence of pfa, slag and microsilica on chloride induced corrosion of reinforcement in concrete. *Cement and Concrete Research*, 21(5), 819-834. doi:10.1016/0008-8846(91)90177-J
- Montemor, M. F., Simoes, A. M. P., & Ferreira, M. G. S. (2003). Chloride-induced corrosion on reinforcing steel: From the fundamentals to the monitoring techniques. *Cement and Concrete Composites*, 25(4-5), 491-502. doi:10.1016/S0958-9465(02)00089-6
- Naito, C., Jones, L., & Hodgson, I. (2011). Development of flexural strength rating procedures for adjacent prestressed concrete box girder bridges. *Journal of Bridge Engineering*, 16(5), 662-670. doi:10.1061/(ASCE)BE.1943-5592.0000186
- Nanayakkara, O., & Kato, Y. (2008). Effect of cathodic-anodic ratio and chloride content on macro-cell corrosion. *Seisan Kenkyu*, 60(3), 20-3.

- Nematollahi, B., Saifulnaz, R. M. R., Jaafar, M. S., & Yen, L. V. (2010). A review on ultra high performance 'ductile' concrete (UHPdC) technology. *International Journal of Civil and Structural Engineering*, 2(3), 1003-18.
doi:10.6088/ijcser.00202030026
- Nishikata, A., Yamashita, Y., Katayama, H., Tsuru, T., Usami, A., Tanabe, K., & Mabuchi, H. (1995). An electrochemical impedance study on atmospheric corrosion of steels in a cyclic wet-dry condition. *Corrosion Science*, 37(12), 2059-69.
doi:10.1016/0010-938X(95)00104-R
- Otieno, M. B., Alexander, M. G., & Beushausen, H. D. (2010). Suitability of various measurement techniques for assessing corrosion in cracked concrete. *ACI Materials Journal*, 107(5), 481-489.
- Perry, V., Moore, B., & Bierwagen, D. (2006). Revolutionary concrete solutions understanding ultra-high-performance, fiber-reinforced concrete. *Construction Specifier*, 59(10), 40-52.
- Pour-Ghaz, M., Isgor, O. B., & Ghods, P. (2009). The effect of temperature on the corrosion of steel in concrete. part 1. simulated polarization resistance tests and model development. *Corrosion Science*, 51(2), 415-25.
doi:10.1016/j.corsci.2008.10.034
- Poursaei, A. (2011). Corrosion measurement techniques in steel reinforced concrete. *Journal of ASTM International*, 8(5), JAI103283 (15 pp.). doi:10.1520/JAI103283

- Ralph, B., Burkert, A., & Gerd, E. (2006). Portable equipment for determination of corrosion stage of concrete structures - comparison of laboratory and on-site experiences. Paper presented at the *Corrosion 2006, September 10, 2006 - September 14*, 064071-0640711.
- Reou, J. S., & Ann, K. Y. (2009). Electrochemical assessment on the corrosion risk of steel embedment in OPC concrete depending on the corrosion detection techniques. *Materials Chemistry and Physics*, 113(1), 78-84.
doi:10.1016/j.matchemphys.2008.07.063
- Shaheen, E., & Shrive, N. G. (2007). Cyclic loading and fracture mechanics of Ductal[®] concrete. *International Journal of Fracture*, 148(3), 251-260.
doi:10.1007/s10704-008-9199-1
- Shi, C., Deng, D., & Xie, Y. (2006). Pore structure and chloride ion transport mechanisms in concrete. Paper presented at the *Environmental Ecology and Technology of Concrete*, , 302-303 528-535.
- Skazlic, M., & Bjegovic, D. (2009). Toughness testing of ultra high performance fibre reinforced concrete. *Materials and Structures/Materiaux Et Constructions*, 42(8), 1025-1038. doi:10.1617/s11527-008-9441-3
- Su-Tae Kang, Lee, Y., Yon-Dong Park, & Jin-Keun Kim. (2010). Tensile fracture properties of an ultra high performance fiber reinforced concrete (UHPFRC) with

steel fiber. *Composite Structures*, 92(1), 61-71.

doi:10.1016/j.compstruct.2009.06.012

Trinh, B. P., & Van Chanh, N. (2012). Study on the properties of ultra high performance fiber reinforced concrete. Paper presented at the *2011 International Conference on Manufacturing Science and Technology, ICMST 2011, September 16, 2011 - September 18, , 383-390* 3305-3312. doi:10.4028/www.scientific.net/AMR.383-390.3305

Tsukahara, E., & Uomoto, T. (2000). Corrosion rate of reinforcing steel bars in cracked concrete. *Transactions of the Japan Concrete Institute*, 22, 155-166.

Ustabas, I. (2012). The effect of capillarity on chloride transport and the prediction of the accumulation region of chloride in concretes with reinforcement corrosion.

Construction & Building Materials, 28(1), 640-7.

doi:10.1016/j.conbuildmat.2011.10.043

Vedalakshmi, R., Balamurugan, L., Saraswathy, V., Kim, S. -, & Ann, K. Y. (2010).

Reliability of galvanostatic pulse technique in assessing the corrosion rate of rebar in concrete structures: Laboratory vs field studies. *KSCE Journal of Civil Engineering*, 14(6), 867-77. doi:10.1007/s12205-010-1023-6

Yoo, J., Park, Z., Kim, J., & Chung, L. (2003). Development of a galvanic sensor system for detecting the corrosion damage of the steel embedded in concrete structures: Part

1. laboratory tests to correlate galvanic current with actual damage. *Cement and Concrete Research*, 33(12), 2057-2062. doi:10.1016/S0008-8846(03)00226-6
- Yunovich, M., Thompson, N. G., & Virmani, Y. P. (2005). Corrosion protection system for construction and rehabilitation of reinforced concrete bridges. *International Journal of Materials & Product Technology*, 23(3-4), 269-85.
doi:10.1504/IJMPT.2005.007731
- Zhang, R., Castel, A., & Francois, R. (2012). Concrete cracking due to chloride-induced reinforcement corrosion - influence of steel-concrete interface defects due to the 'top-bar effect'. *European Journal of Environmental and Civil Engineering*, 16(3-4), 402-413. doi:10.1080/19648189.2012.667984
- Zhang, R., Castel, A., & Francois, R. (2009). The corrosion pattern of reinforcement and its influence on serviceability of reinforced concrete members in chloride environment. *Cement and Concrete Research*, 39(11), 1077-1086.
doi:10.1016/j.cemconres.2009.07.025
- R. F. Stratfull, *Half-cell potential and the corrosion of steel in concrete*. 1972, California division of highways, State of California: Sacramento, California.
- ACI 222R, *Corrosion of Metals in Concrete*. 1996
- Hansson C. M. , *Comments on electrochemical measurements of the rate of corrosion of steel in concrete*. *Cement and Concrete Research*, 1984. **14**: p. 574-584.

Paul Acker, Ductal Technology: A LaFarge Spectrum of Properties, A Wide Range of Applications, Paris, France.

C. M. Hansson, A. Poursaei and S. J. Jaffer (2007), CORROSION OF REINFORCING BARS IN CONCRETE, Portland Cement Association.

Denny A. Jones (second edition), Principles and Prevention of Corrosion.

David Darwin, JoAnn Browning, Matthew O'Reilly, Carl Locke Jr. and Y. Paul Virmani (2011), Multiple Corrosion Protection Systems for Reinforced Concrete Bridge Components, Federal Highway Administration.