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SYNTHESIS GUIDE TO BEST PRACTICES FOR CORROSION RESISTANT CONCRETE

Prepared For:

Utah Department of Transportation
Research Division

Submitted by:

University of Utah
Department of Civil and Environmental Engineering

Authored by:

Paul J. Tikalsky, Ph.D., P.E.
David E. Beh

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16. Abstract

This document provides guidelines for the design and construction of corrosion resistant reinforced concrete structures with 75 to 100 years of design life. This level of durability is considered essential for environmentally sustainable solutions and for the economic viability of a long-life design strategy. It is also consistent with the “Highways for Life” policy of the US federal government. The project considered the new and existing technologies in tension reinforcement for concrete structures. These included dual phase steels, epoxy coated bars, fiber reinforced polymer bars and different forms of stainless steel. The resulting guidelines for long life bridges include the best practices from FHWA and other states, as well as, new practices and technologies that increase the life-cycle cost and longevity of highway structures. The cost of performance based concrete mixtures and reinforcing materials were considered in the evaluation of the alternative solutions. The value of using performance based specifications for concrete and corrosion resistant reinforcing steel or fiber reinforced polymers is shown to be both cost efficient and environmentally sound policy. Designing concrete with low permeability and moderate shrinkage prevents the ingress of deleterious ions and moisture. The suggested changes to the Utah DOT specifications are noted in the appendices.

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EXECUTIVE SUMMARY

This document provides guidelines for the design and construction of corrosion resistant reinforced concrete structures with 75 to 100 years of design life. This level of durability is considered essential for environmentally sustainable solutions and for the economic viability of a long-life design strategy. It is also consistent with the "Highways for Life" policy of the US federal government. The project considered the new and existing technologies in tension reinforcement for concrete structures. These included dual phase steels, epoxy coated bars, fiber reinforced polymer bars and different forms of stainless steel. The resulting guidelines for long life bridges include the best practices from FHWA and other states, as well as, new practices and technologies that increase the life-cycle cost and longevity of highway structures. The cost of performance based concrete mixtures and reinforcing materials were considered in the evaluation of the alternative solutions. The value of using performance based specifications for concrete and corrosion resistant reinforcing steel or fiber reinforced polymers is shown to be both cost efficient and environmentally sound policy. Designing concrete with low permeability and moderate shrinkage prevents the ingress of deleterious ions and moisture. The suggested changes to the Utah DOT specifications are noted in the appendices.

CHAPTER 1. INTRODUCTION

Early cracking of concrete and corrosion are a major concern for all transportation agencies. The causes and mitigation alternatives are key aspects of creating and maintaining a sustainable transportation infrastructure. Utah has approximately 2850 bridges. A recent survey by the FHWA has identified 19 percent of Utah's bridges are structurally deficient or functionally obsolete. In some cases, the bridges have simple unlived their usefulness or the existing expectations are greater than the remaining design capacity. There is an increased concern about the integrity of the aging infrastructure throughout the US and Utah has to look forward to ensure that future designs of highway structures are durable structures with long design lives. As such it is important to develop design guidelines that result in efficient, economical, corrosion resistant transportation structures. This report addresses the design issues that impact the initiation, propagation, mitigation or elimination of corrosion of steel in highway structures.

Corrosion in transportation structures is an electrochemical process that provides for the oxidation and eventual reduction of structural reinforcing or prestressing steel. There are two conventional methods for mitigating corrosion: 1) increase the time period in which the steel is protected from a corrosive environment and, 2) increase the time associated with the propagation of corrosion. Concrete is a highly alkaline environment that passively protects steel from the oxidation/reduction reactions of corrosion. Undisturbed in a moderate environment, reinforcing steel in a concrete structure may last for one hundred years or more. However, highway structures are subject to cracking from loading, freeze-thaw cycles, early age construction conditions, as well as, deicing chemicals, and a variety of other physical and environmental conditions. Deicing chemicals, particularly chloride salts create chloride concentrations that diffuse into concrete and reach the reinforcing steel, destroying the passive protection of the alkaline concrete. Increasing the time to exposure of detrimental chloride concentrations can be achieved by reducing the permeability and diffusion properties of the concrete, controlling the size and distribution of cracks, and by the use of certain chemical admixtures in concrete. The other half of the equation is corrosion propagation. Once conditions exist to corrode steel, the speed at which the reaction occurs depends on the amount of steel surface exposed to the reactive environment and the resistance of the materials in the reactive circuit. The propagation of

corrosion can be severely slowed by protective coatings over the steel, corrosion resistant alloy steel (stainless or MMFX™) or by increasing the electrical resistance of the corrosion cell.

Utah is a unique environment with varying climates, from sulfate rich soils in the south to extreme annual temperature differentials along the Wasatch front. The entire state is subjected to extremely low humidity and snowy conditions. High quality concrete can be designed to meet the demands of harsh Utah climates and can be produced and placed economically through properly controlled concrete operations. The proper design of concrete structures using the correct materials specific to each application will create longer lasting structures which will in turn reduce the number of times the structure will need to be rebuilt. Corrosion resistant concrete is essential for such sustainable structures. The recommendations included in this report will assist in maintaining the Utah Department of Transportation's leadership among transportation agency.

Objective

The primary objective of this document is to provide guidelines for the design and construction of corrosion resistant reinforced concrete structures with 75 to 100 years of design life. The guidelines for long life bridges will include the best practices from FHWA and other states, as well as, new practices and technologies that increase the life-cycle cost and longevity of highway structures.

CHAPTER 2. FHWA SURVEY

2.1 INTRODUCTION

The transportation community is making great advancements with implementation of High Performance Concrete (HPC) technology in an effort to extend the service life of pavements and bridges over the past ten years. During the fall of 2003 and winter of 2004, the FHWA High Performance Concrete (HPC) Technology Delivery Team conducted a survey and compiled results on the status of HPC implementation nationwide. All 50 states, the District of Columbia, Puerto Rico and the FHWA Federal Lands Highway Division Bridge Office responded to a recent survey that they have incorporated HPC specifications in projects involving bridge decks, superstructures and/or substructures. These projects took advantage of the high strength and/or high durability attributes of HPC. The results are available on the web. In total, this is a comprehensive look at the national effort underway to implement HPC bridge technology over a 10-year plus period. The data was updated in 2006.

2.2 Survey

Many states include a section or special provision on high-performance concrete (HPC) for their specifications. HPC has been defined by ACI as concrete that meets special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing and curing practices (ACI 116R). Because of the broadness of this definition, each states understanding of what HPC entails may be slightly different making an exact survey of what each state is currently doing somewhat difficult. Some of the questions in this survey bear directly on the state-of-the-practice in the US on corrosion resistant concrete.

The third question of the survey is related to the type of distress that the agency is most commonly experiencing. The responses to this question revealed that early cracking and corrosion of the reinforcing steel were the most frequent forms of distress. While all types of cracking are a concern to DOTs, early age cracking usually results in wider cracks of greater frequency. The width of the crack plays a particularly important role in the ingress of deicing salts and moisture into the concrete and eventually to the level of the reinforcing steel. Many of the state DOTs also stated that corrosion of reinforcing steel is an important factor in choosing

HPC for concrete specifications. The National Bridge Survey (NBS) shows that the many of the structurally deficient and functionally obsolete bridges in the US have corrosion related issues. Some of these issues are related to seawater exposure and others from deicing salt exposures.

The fourth question of the survey concerned the admixtures used and the curing requirements of the agency to mitigate distress. The responses to this question revealed that the great majority of the states were making efforts to mitigate concrete deterioration and corrosion with the use admixtures and/or proper curing techniques. This includes the use of corrosion inhibiting admixtures by several states, and the move to permeability reducing admixtures in precast elements in other states (box culverts, p/s beams and boxes, etc.). Still other states have tried using lithium admixtures to reduce ASR cracking that may accelerate the ingress of salts over time. Nearly all states prohibit the use of chloride based accelerators because of their tendency to increase the frequency of corrosion at early ages. Curing practices vary throughout the country vary widely from state to state. This is also true of w/cm ratio requirements and the permissibility of contractors to add water at the site. Most of the northern tier states that have implemented any form of HPC use wet curing procedures that vary from 7 to 14 days. The most pertinent discussion that exists in the FHWA HPC implementation task group, TRB and ACI is that 7-days wet curing is the recommended minimum for quality concrete, although this may not be practical in long paving operations. Removing curing measures should be conducted over several days; reducing or eliminating the water source at the ending of the wet cure period and removing the burlap or cotton mats several days later. The immediate removal of curing measures has been known to cause hygral and/or thermal shock, leading to early age cracking. With respect to w/cm ratio, TRB and ACI technical committees recommend a number between 0.40 and 0.48 for quality concrete that is designed to resist corrosion. The current state-of-the-practice is to not allow substantial additions of water after it has left the plant. This reduces the number of mistakes and quality issues in future years. Trucks are delivered with batch tickets that give a range of water that would be permitted to be added to the concrete and still meet the w/cm specifications given by the engineer. According to the survey, most states will allow water to be added to trucks delivered to the job site as long as w/cm are still in the permissible range set by the engineer. However the states with HPC specification, severely limit this practice.

The fifth question of the survey asked whether the agency had used fiber- reinforced concrete for bridge decks or overlays and whether that fiber- reinforcing was standard practice or

experimental. Several states have used fiber reinforcing experimentally, but few states use fiber-reinforcing regularly. While there are many types of fiber on the market, some types of polymer fiber and glass fiber reinforcement, in addition to smaller diameter reinforcing bars, will greatly reduce cracking and crack width and thereby reduce chloride ion ingress.

Question 6 of the survey concerns reinforcement specifications for aggressive and non-aggressive environments in addition to experimentally used reinforcements. Most states were specifying epoxy coated reinforcement for aggressive environments. Some states only specified ECR for the top mat and black steel for the bottom mat; while other specified only black bar. No states were regularly using stainless steel or stainless steel clad reinforcing bars. There have been recent publication by researchers in Virginia that ECR is not economically viable in states with moderate climates and salt usage, that increasing the resistance to chloride ingress may be a better value.

The tenth question of the survey was perhaps the most useful question in determining what states are doing in order to reduce cracks and corrosion. The question concerned the QA/QC measurements taken by the agency. While many states are still specifying concrete with only slump, air content, and compressive strength, several states have implemented quality control measures in order to address issues faced for specific environments. The use of AASHTO T277, chloride ion permeability test, ASTM C157, shrinkage potential of concrete mixtures, and maturity have all been implemented in one or more states to better reduce the ingress of salts into concrete. UDOT has specified chloride ion permeability requirements for the some areas in the Legacy Highway project.

Question 10 may actually be the greatest indication of an agency's commitment to the concrete that is placed. Several agencies specify their concrete by compressive strength, slump, and air content. These are all important properties for concrete, however, there are other factors that affect the durability of the concrete.

Question 12 of the study requested the reasons that state DOTs use or have tried HPC. Most states are using HPC in order to reduce permeability and increase the durability of their highway infrastructure. This is a long-term view and one that focuses on life-cycle costs. However, some states are using high strength concrete, a form of HPC, to reduce the number of prestressed girders or to increase the span length.

2.3 LESSONS FROM OTHER STATES

This comprehesive survey of state DOTs revealed some very innovative transportation agencies and specifications. These innovations and specification improvements are laying the groundwork for future states to build upon.

Virginia and New York have been some of the most aggressive states in implementing HPC standard that improve their highway infrastructure. Virginia has worked with the VTRC to implement performance based specification and pushed into ways to encourage accountability from the contractors. Virginia DOT has used HPC with penalty clauses for concrete not meeting the specifications. The contractor was required to made mock ups to ensure that concrete was meeting specifications. VTRC has been involved in research of alternative reinforcements for several small projects, as well. New York has used FHWA and other work to prescribe concrete mixture designs that meet durability standards, primarily through the use of silica fume and fly ash.

California's has developed concrete performance based specifications. This specification has resulted in a moderately high use of pozzolans (fly ash, slag and silica fume). California DOT's specification does not specify QC tests unless there is a specific need. The state has also has done projects with alternative reinforcement materials. The Pennsylvania DOT has constructed some 40 bridges with HPC specifications that require both mandatory (e.g. permeability, shrinkage, strength, hardened air content, ASR resistance) and informational testing (e.g. scaling, sulfate, freezing and thawing resistance). Both of these states reduced their greenhouse emmisions related to concrete construction and experienced higher quality of concrete.

Colorado was involved in the use of HPC from the beginning of a study by the FHWA and the Strategic Highway Research Program (SHRP). Like Pennsylvania, Colorado is requiring chloride permeability and shrinkage tests. CDOT has also participated in FRP reinforced experimental projects. Interstate 25 over Yale Ave. in Denver utilized HPC to reduce the depth of the girders in order to maintain clearance without changing the grade. New York has been specifying HPC for decks for more than ten years through the use of a prescriptive specification that has been shown to deliver high performance characteristics.

Delaware has very tight quality control testing for transportation projects and has been involved in a number of projects using alternative reinforcing materials. Florida DOT has made

significant contributions to the understanding of alternative reinforcement through extensive testing. The Florida DOT, in conjunction with the University of Florida, has constructed a special exposure facility for half-cell potential reading of steel in seawater environments. Also, they have developed several special QC tests are now standard practice in Florida. Oregon has had many projects using alternative reinforcement and has moved away from specifying ECR for marine structures.

Texas Louetta Overpass state highway 249 in Houston US route 67 over North Concho River used high strength and low permeability concrete with uncoated reinforcement for a post tensioned box structure. Creep and shrinkage were specifically monitored for these projects. Several other states are working on HPC related issues and future implementation. Washington DOT has been collecting data on life-cycle costs for HPC, while West Virginia has implemented a requirement for test slabs for all new mixture, monitoring evaporation rates and permeability reduction with time.

Most agencies have requirements for strength, slump and total air content. In addition, almost all agencies require certain water cement ratios and limit the amount of supplementary cementitious material. A better approach would be performance based design specifications that allow the designers to define the requirements of the application and environment and the ready mix suppliers and the contractor more technical options to use materials that meet these design requirements. This is not easily implemented without trust in the technical expertise of the ready mix suppliers or a contractual warrantee that the concrete will perform as directed.

CHAPTER 3. PERFORMANCE BASED DESIGN

3.1 DEFINING PERFORMANCE

Performance based design requires an understanding of the structural and environmental demands of the structure or pavement over time. Most state agencies are still specifying concrete by slump, air, and 28-day design compressive strength. On top of these specifications, most states enforce minimum cement contents and maximum water/cement ratios. With an increased attention to the aging infrastructure, there has been an increased focus on designing 75 to 100 year structures. Proper design, construction, and maintenance of concrete transportation structures will result in lower life cycle costs, and in many cases, lower initial costs.

The term HPC is used to describe concretes that are made with carefully selected high quality ingredients, optimized mixture designs, and which are batched, mixed, placed, consolidated and cured to the highest industry standards. It is not unusual for HPC mixtures to have a water-cementitious materials ratio (w/cm) between 0.40 and 0.45, like that of existing specifications. However the optimization of the portland cement, pozzolans, aggregate gradations, and admixtures may result in a less expensive mixture with greater durability characteristics.

Several definitions have been proposed over the years to familiarize the engineering community and concrete industry with HPC. HPC is defined by ACI as concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing and curing practices. More recently, the National Concrete Bridge Council has drafted a definition for HPC as, "…concrete that attains mechanical, durability or constructability properties exceeding those of normal concrete." In the future, we may just refer to this concrete as concrete that performs as designed.

HPC, as defined above, can lead to more efficient longer lasting structures. This will help agencies preserve both natural and economic resources and create a more sustainable infrastructure. Properly designed concrete is an essential versatile durable construction material that will reduce construction waste and optimize materials leading to a more environmentally friendly transportation infrastructure.

Performance characteristics defined as durability include freeze-thaw resistance, scaling resistance, abrasion resistance, chloride ion penetration, alkali-silica reactivity, and sulfate

resistance. Structural design performance characteristics include compressive strength, modulus of elasticity, shrinkage, and creep. The characteristics are determined using standard test procedures, and grades of performance are suggested for each characteristic. Durability is often just as important as the structural characteristics for aggressive environments and requires strict quality control measures.

The material characteristics and performance grades should be selected in accordance with the intended application and the concrete's environment. For example, a bridge deck supported on girders needs a specified compressive strength but is unlikely to require specified values for modulus of elasticity and creep. The characteristics specified will depend on the environmental factors affecting the structure and loads. It is not necessary to require every imaginable performance characteristics for a given application. Only those technical characteristics that are required should be specified. Other important features of HPC are uniformity and consistency. With higher expectations and optimized constituents, there is a need to decrease the variability in the concrete produced and special attention is required for producing consistent concrete.

3.2 CONCRETE PERFORMANCE STANDARDS

The first step for a performance based design is defining the needs that will be fulfilled by the concrete structure or pavement. This will require that the agency understand the traffic loading, environmental factors and design constraints of the structure (Tepke & Tikalsky, Best Construction Practices for Concrete Bridge Decks, 2007).

After the requirements are fully understood, these needs are translated into equivalent design loads and environmental exposures. Defining the design requires knowledge about the application, desired design life, availability of materials, the structural requirements and the material properties. Materials, pavement and structural engineers should be communicating about the design requirements and the available alternatives that meet the design definitions.

Materials pavement and structural engineers will then need to decide on the performance specifications. Guidelines can be established like those shown in Table 3.1, developed for the Pennsylvania Department of Transportation (Tepke & Tikalsky, Best Construction Practices for Concrete Bridge Decks, 2007). It should be noted that not all performance criteria need to be specified for every project. For example, sulfate resistance would not need to be specified for

bridge decks and freeze-thaw durability would need to be specified for such elements exposed to moisture and freezing temperatures.

Bridge decks reinforced with plain carbon steel in an environment subject to freeze-thaw cycles, and deicing salts would be best protected with low permeable concrete with minimal cracking and scaling. For such applications, engineers might specify chloride permeability less than 1500 coulombs, freeze-thaw durability greater 90%, scaling resistance visual rating less than 2, and shrinkage less than 500 microstrains at 56 days. These goals might be met through ternary blends, properly entrained air, proper finishing and curing techniques. Fiber reinforcement could also help control the width of cracks.

Table 3.1 Grades of performance characteristics for high performance structural concrete

Performance characteristic	Standard test method	HPC performance characteristic grade		
		1	2	3
Freeze-thaw durability (F/T=relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666 Proc. A	70%≤F/T<80%	80%≤F/T<90%	90%≤F/T
Scaling resistance (SR=visual rating of the surface after 50 cycles)	ASTM C 672	3.0≥SR>2.0	2.0≥SR>1.0	1.0≥SR≥0.0
Abrasion resistance (AR=avg. depth of wear in mm)	ASTM C 944	2.0>AR≥1.0	1.0>AR≥0.5	0.5>AR
Chloride penetration (CP=coulombs)	AASHTO T 277 ASTM C 1202	2500≥CP>1500	1500≥CP>800	800≥CP
Alkali-silica reactivity (ASR=expansion at 56 d) (%)	ASTM C 441	0.20≥ASR>0.15	0.15≥ASR>0.10	0.10≥ASR
Sulfate Resistance (SR=expansion) (%)	ASTM C 1012	SR≤0.10 at 6 months	SR≤0.10 at 12 months	SR≤0.10 at 18 months
Flowability (SL=slump, SF=slump flow)	AASHTO T 119 ASTM C 143, and proposed slump flow test	SL>190 mm (SL>7-1/2 in), and SF<500 mm (SF<20 in)	500≤SF≤600 mm (20≤SF≤24 in)	600 mm<SF (24 in<SF)
Strength (f'_c=compressive strength)	AASHTO T 22 ASTM C 39	55≤f'_c<69 MPa (8≤f'_c<10 ksi)	69≤f'_c<97 MPa (10≤f'_c<14 ksi)	97 MPa≤f'_c (14 ksi≤f'_c)
Elasticity (E_c=modulus of elasticity)	ASTM C 469	34≤E_c<41 GPa (5≤E_c<6x10^6 psi)	41≤E_c<48 GPa (6≤E_c<7x10^6 psi)	48 GPa≤E_c (7x10^6 psi≤E_c)
Shrinkage (S=microstrain)	AASHTO T 160 ASTM C 157	800>S≥500	500>S≥300	300>S
Creep (C=microstrain/pressure unit)	ASTM C 512	75≥C>55/MPa (0.52≥C>0.38/psi)	55≥C>30/MPa (0.38≥C>0.21/psi)	30/MPa≥C (0.21/psi≥C)

CHAPTER 4. EFFECT OF MATERIALS ON CORROSION

4.1 INTRODUCTION

The concrete mixture design and cementitious material selection is important in protecting reinforced concrete from corrosion. The choice may determine the time required for the chlorides to diffuse to the level of the reinforcing in a sufficiently high concentration to initiate corrosion. There are many different items that should be considered when choosing the appropriate cementitious materials for a specific project. The technical value of trial batches and pre-construction testing cannot be overemphasized. Cementitious materials should have a reasonable fineness to avoid a high water demand, unless there is some value to high early strength. Fineness of cementitious materials varies according to source and processing. Generally, finer cementitious materials react sooner and demand more water for the same mobility. Fine portland cement with high alkali contents have been known to compound ASR problems and those high in alumina have been known to compound sulfate attack problems. However, fine low calcium fly ashes greatly reduce ASR and sulfate problems.

Pozzolans and a special class of cementitious materials that use the excess calcium hydroxide from the reaction of portland cement to generate more cementing compounds. The result of such reactions is a refined matrix of cementing compounds that has a variety of desirable qualities. There are several advantages to using pozzolans with portland cement in concrete:

- Reduced permeability
- Reduced cost
- Lower heats of hydration
- Increased resistance to ASR
- Increased resistance to sulfate attack
- Lower carbon footprint
- Higher long-term compressive strengths
- Increase workability during construction
- Reduced impact on the solid landfill of the community

It should be noted that each pozzolanic materials provides its own benefits. High calcium fly ash is different from low calcium fly ash and silica fume and slag are not the same as either fly ash.

4.2 SUPPLEMENTARY CEMENTITIOUS MATERIALS

One of the most important aspects of designing HPC mixtures is the optimization of the cementitious materials. The primary cementitious material in most highway applications is portland cement. Portland cement is inexpensive hydraulic cement manufactured at high temperatures from limestone, clay and gypsum. Pozzolans and slag cement is added to concrete mixtures with portland cement as "supplementary cementitious materials (SCM)." While some SCMs are hydraulic cements by themselves, they typically work much better when blended with portland cement. The technical advantages of SCMs are listed in the previous section, however they also provide a potential economic advantage. Fly ash and natural pozzolans typically cost less than portland cement and when optimized the total cost of portland cement and these materials is likely less than a mixture with portland cement alone.

Proper water to cementitious materials ratio is also very important in producing concrete that is resistant to corrosion. Lower w/cm results in less internal voids in the concrete and a more compact microstructure in the concrete paste. This refined microstructure has a lower diffusion constant and a higher resistance to electrical current. Both of these properties are advantageous to protecting steel from the effects of corrosion. The use of some pozzolans or SCMs may add workability to the concrete and require less total water. This is especially true for fly ashes. At the very least, different dosages of water reducing admixtures and other admixtures will be required when using SCMs. The dosages cannot be linearly reduced or increased because SCMs require different amounts of admixture for the same effect. To determine the dosages and the interaction of these materials with portland cement, the designer or ready-mix supplier needs to prepare trial batches to ensure material compatibility through the wet and hardened concrete properties.

Silica fume, or microsilica, is a byproduct of silicon-metal production. Silica fume is much finer grained than portland cement with an average diameter is 0.1 μm (0.004 mils). Silica fume generally contains over 90% silicon dioxide and has a specific gravity in the range of 2.10 to 2.55 (American Concrete Institute, 2007). Silica fume concrete was first used in highway applications in the United States in the mid-1980s. Since that time, the use of silica fume concrete has grown considerably. Silica fume is typically used as a small percentage of total cementitious materials, e.g. 3-8 percent of total cementitious material. It costs six to ten times

the cost of portland cement and therefore must be used judiciously to be economical. Its fine particle size often increases water demand, but also allows it to enter into the cementitious reactions sooner that other materials. It is recommended that silica fume concrete be made with a high-range water-reducing admixture (HRWRA). Concrete containing silica fume rarely bleeds and therefore it must be wet cured from the time of placement to prohibit early age cracking.

The effects of silica fume in concrete are to lower the permeability and diffusion constants, increase early age strength, delay the onset of ASR and provide a sticky adhesive characteristic to fresh concrete. These characteristics are ideal for protecting steel from a corrosive environment. The sensitivity of concrete containing silica fume to curing conditions and early age cracking is a negative for creating a corrosion resistant environment. Although these can be control through good concrete practices, the potential exists for early age cracking which is a direct pathway for chlorides and moisture.

Currently there are no provisions in UDOT's standards and specifications for slag cements. Utah may be one of only a few states that does not provide for such supplementary cementitious materials. While there is not a source in the immediate vicinity of Utah, the changes in market conditions may make it available. Slag cements (ASTM C989) provide for a reduction in permeability, strength developments similar to Type II portland cement, high volume replacements for high greenhouse gas reductions, reduced impact on solid landfills, and excellent resistance to sulfate attach and moderation of ASR.

Blast furnace slag is a byproduct of iron production. When the slag is ground and granulated shortly after being produced it is both a hydraulic cement and a pozzolan. It requires only about 3 percent of the CO_2 to manufacture a slag cement as it does to manufacture a portland cement. Slag is typically 95% silicates, aluminates and calcium. It can be used as a portion of the cementitious material in concretes with proportions ranging from 35 ~ 70 percent by mass of total cementitious material. Slag cement inclusion reduces heat evolution, environmental impacts, and susceptibility to ASR and sulfate attack. The addition of GBFS may also reduce the amount of HRWRA required to attain the same flowability as a mixture containing only portland cement. Creep and shrinkage of slag containing concretes are not significantly different than concretes not containing cements.

4.3 BLENDED CEMENTS

Blended cement, as defined in ASTM C 595, is a mixture of portland cement and blast furnace slag (BFS) or a mixture of portland cement and a pozzolan. UDOT has provisions allow for the use of some types of blended cements (Section 03055 Part 2.2 C). The use of blended cements in concrete reduces the variability in handling and blending cementitious materials at the ready-mix plant. The blended cements have also been optimized for the most efficient SO_3 content. This is a key component in controlling setting time and compressive strength development. Blending multiple cementitious materials at the ready mix-plant by adding fly ash, slag cements, silica fume, natural pozzolans or metakaolin to the portland cement can often produce suboptimal strength development, finishing and setting properties. The blending process can be done with ball mills, air blenders, or mechanical blenders. Each has been shown effective in producing uniform blended cements.

The important consideration with blended cements is that they substantially increase the time to initial corrosion and increase the resistivity of concrete to slow the propagation of corrosion; much in the same manner as the combination of portland cement and pozzolans. The difference is in the uniformity of blended cements and larger variability in mixtures blended at the ready-mix plant.

4.4 PERFORMANCE CEMENTS

Performance cement, as defined in ASTM C 1157, "classifies cements by type based on specific requirements for general use, high early strength, resistance to attack by sulfates, and heat of hydration. Optional requirements are provided for the property of low reactivity with alkali-reactive aggregates." There is no general restriction on the composition of the hydraulic cements. Such hydraulic cements are typically blends of portland cement, pozzolans, slags, finely divided limestone particles and other mineral additives.

UDOT has provisions allow for the use of limited performance cements (Section 03055 Part 2.2 C). The use of performance cements in concrete reduces the variability in handling and blending cementitious materials at the ready-mix plant and allows for the design of classes of cement for specific applications. The performance cements have been optimized for the most efficient SO_3 content and with the chemical resistance desired by the specifying agency. These are a key components in creating a durable infrastructure. Blending multiple cementitious

materials at the cement plant under controlled conditions with fibers, fine limestone particles and powdered admixtures can often produce optimal strength development, durability, finishing and setting properties. The blending process can be done with ball mills, air blenders, or mechanical blenders. Each has been shown effective in producing uniform performance grade cements. The performance cements can be designed to create corrosion resistant concrete through the use of low permeability, low shrinkage standards as well as the potential addition of corrosion inhibitors. The difference is in the uniformity of blended cements and larger variability in mixtures blended at the ready-mix plant.

There is another class of performance based cement known as "shrinkage compensating cements." These cement reduce the number of cracks and the width of the cracks to inhibit the intrusion of chlorides and water into the concrete. There are 3 different types of shrinkage compensating cements Type K, M, and S as classified by ASTM C845. The 3 types are classified separately by the form of aluminate compounds from which the expansive ettringite is formed. These cements can offset the early shrinkage caused by improper curing and autogenous shrinkage by slightly expanding during the initial days of hydration. There are some disadvantages associated with using shrinkage compensating cement. The cements tend to be expensive and have limited availability. The diffusion constants of these types of cement are similar to plain portland cement concrete, rather than that of blended cements. However, the mitigation of cracking can be very important in some structures and therefore, they can have a positive effect on the mitigation of corrosion in concrete structures.

4.5 CHEMICAL ADMIXTURES

There are several types of admixture which influence the corrosion of steel in reinforced concrete. These admixtures are used for a variety of technical reasons and in most cases can be easily screened to manufacture concrete that either has no detrimental effects from the admixtures or contains admixtures that inhibit corrosion. In general, a few large classes of admixtures have very limited effects on the initiation or propagation of corrosion in reinforced concrete. Air entraining agents and most water reducing admixtures have little effect on the corrosion mechanism of plain carbon steel. While each of these admixtures may reduce the w/cm ratio this results in a small reduction in the permeability/porosity of the concrete. High range water reducers (HRWR) likely have the greatest effect, but even this effect is marginal

Permeability reducing admixtures can reduce the rate at which moisture and aggressive chemicals penetrate concrete, but cannot prevent it completely. These types of admixtures come in many proprietary formulations. Some of these admixtures contain latex formulations, others are microsilica or sodium silicate based and still others are surfactants or oil based. The surfactants can entrain air into the concrete matrix which will reduce compressive strength.

There are basically 3 different corrosion inhibiting admixtures that are commercially available: select nitrite based compounds, lignosulfates, and amines and esters based admixtures. Calcium nitrite may be the most common nitrite base compound. Nitrites react with ferrous ions at the surface of the steel to create a passive layer on the reinforcing steel. The nitrites block the current path between steel layers to shut down the galvanic pathway that is necessary to complete the oxidation-reduction circuit. Nitrite based admixtures also help develop early strength. Although not as commonly used as corrosion inhibitors, the lignosulfates have shown the ability to lessen the initiation of corrosion. Lastly, the amines and esters compounds develop organic coatings on the steel and reduce the rate of chloride penetration through the concrete matrix. These compounds are not fully developed but may hold promise in years to come.

CHAPTER 5 AGGREGATE

5.1 EFFECTS OF NORMAL AGGREGATES ON CORROSION

Quality materials are essential to long lasting structures and aggregate is no different. Aggregates play a small, but important role in manufacturing concrete that is resistant to corrosion. The quality of the aggregate determines its susceptibility to cracking and its compatability with the paste structure. For concrete to develop strong bonds between aggregate and paste, the aggregates must be clean and free of debris. Keeping the maximum aggregate size as large is permissible with the constraints of cover requirements, spacing between bars, and element thickness will reduce the shrinkage and cracking. In addition to aggregate size, aggregate proportioning and gradation play a role in manufacturing corrosion resistant concrete. Higher proportions of coarse aggregates that are well graded will reduce autogenous and drying shrinkage.

A fine aggregate with a fineness modulus greater than 2.6 can be a valued material. With the addition of pozzolans like fly ash, silica fume or slag cement there is often an abundance of fine material in the concrete mixture. A higher fineness modulus sand will contain larger size particles to better fill in the middle sizes. This leads to better compaction and less water demand which was identified earlier in this report as a marginal improvement in corrosion resistance.

5.2 EFFECTS OF LIGHTWEIGHT AGGREGATES ON CORROSION

Structural lightweight aggregates have not been reliably reported to either increase or decrease the resistance of the system to corrosion. The use of lightweight concrete may reduce the modulus of elasticity and thereby reduce the tendency of a horizontal elements to crack. While the true value of lightweight concrete is to reduce cracking and to reduce the structural mass and therefore allow designers to design smaller beams, and increase the span of bridges.

CHAPTER 6 REINFORCEMENT

6.1 INTRODUCTION

The fundamental fact that plain carbon steel is used to reinforce concrete is the premise by which we need to create concrete that is resistant to corrosion. Steel is used to reinforce concrete because they have many complementary characteristics. Steel is efficient in tension and concrete is efficient in compression. Concrete has a pH of 12.6 to 13 and steel is passivated from corrosion at pH values in access of 10. The coefficient of steel and concrete are of the same magnitude reducing the strains from temperature compatibility. Both materials are relatively easy to work with and can be mass produced anywhere in the world. In spite of all these characteristics, steel corrosion is the major cause of structural deficiency in bridges and structures and corrosion is a multi-billion dollar issue in the nation's transportation infrastructure.

6.2 CORROSION MECHANISM

As stated in the introduction, the steel in reinforced concrete structures is passivated by the alkaline environment in concrete. A thin oxide film forms on the steel in the highly alkaline concrete pore water and prevents the steel from oxidizing. The alkaline environment is primarily maintained by the sodium, potassium, and calcium in the pore water within concrete. The penetration of chloride ions or the carbonation of calcium hydroxide within concrete decreases the alkalinity over time and subsequently destroys the passivation film. As shown in Figure 6.1, once this film is broken, oxygen and moisture reach the steel and the corrosion reaction begins, using the remaining passivated areas as cathodes and the broken film area as the anode.

Figure 6.1 Corrosion Mechanism

Corrosion is an active chemical process that does not progress at a uniform rate. Changes in environmental conditions such as, moisture, salt concentration, temperature, and electrical current may accelerate or decelerate the rate of corrosion. In plain carbon

reinforcing steel this type of corrosion is called macrocell corrosion. This is a condition where both the anode and cathode of the cell are part of the same alloyed material. The layered pearlite structure in plain carbon steel has carbide as the cathode and ferrite as the anode. The resulting reactive cell results in iron releasing 2 electrons, $Fe \rightarrow Fe_2+ + 2e-$. The two electrons created in the anodic reaction are consumed in a cathodic reaction with water and oxygen, $2e- + H_2O + ½ O_2 \rightarrow 2OH$. The flow of electrons between the anodic and cathodic areas through the steel and its counter-current flow through the concrete pore solution completes the corrosion circuit. The counter flow consists of negatively-charged hydroxide ions and positively-charged ferrous ions. If the concrete's electrical resistance to these ions is high, the rate of current flow carried by the ions will be low. Subsequently, the anodic and cathodic reactions will proceed slowly and the rate of corrosion will be low. The addition of pozzolans was an example of a means of increasing the electrical resistance.

The passive layer provides protection, but can be destroyed. Depassivation may occur under two specific main sets of conditions: (1) reduction of the pH below 10 due to reaction with atmospheric CO_2 (carbonation); or (2) penetration of chloride ions into the concrete pore solution at the level of the steel. Once depassivation occurs, the steel is no longer protected and corrosion may be initiated. CO_2 or CO from the environment or chlorides from deicing salts or seawater diffuse into the concrete over time and react with the hydroxide and calcium ions in the pore solution. Figure 6.2 shows a carbonated section that has been sprayed with phenothylene. The pink section is high pH and the top section is the lower pH from the carbonation. Even when the concrete pore water solution pH level remains high, chloride ions in high concentrations can still effectively depassivate the steel. Chloride ions may diffuse into the concrete or be introduced in the concrete mixture from an admixture, such as the accelerator $CaCl_2$ or in chloride-contaminated aggregates or mixing water. When carbon dioxide molecules penetrate into reinforced concrete, it reacts with solid calcium hydroxide, C-S-H, and alkali and calcium ions in the pore solution and decrease the alkalinity of the pore solution. This reaction creates carbonates and water which evaporates, causing carbonation shrinkage and may create microcracks that permit further carbon dioxide ingress. Carbonation usually penetrates

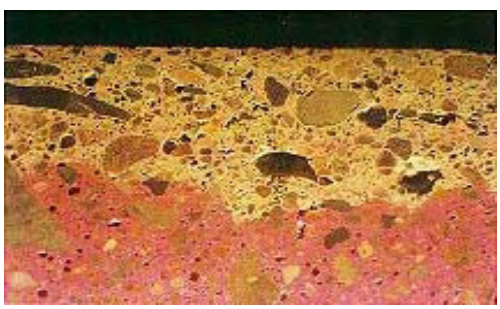

Figure 6.2 Carbonated Concrete

slowly into the concrete member to the level of the reinforcing steel. The time it takes this front to reach the steel is a function of the depth of the cover and of the rate of diffusion of the atmospheric CO_2 or CO into the concrete.

6.3 SERVICE LIFE OF REINFORCED CONCRETE

Service life of a structure or the reinforcing steel in a structure can be illustrated by the model shown in Figure 6.3. It is comprised of an initiation stage (time of completion to time chloride threshold is reached and initial oxidation takes place) and the propagation stage (after initial oxidation to the end of service life). The initiation stage is dependent on many variables, including environment, chloride exposure, cover, and concrete type. In poor or cracked concrete, the initiation stage may be a matter of years, however in well constructed concrete it may be decades and in HPC it can be a century of more. The length of the propagation phase depends on the corrosion rate after the chloride threshold is reached. Corrosion rate may vary considerably depending on the resistivity of the concrete, the oxygen and moisture availability, alloy of the steel, and the environmental conditions. The end of service life is defined by the user.

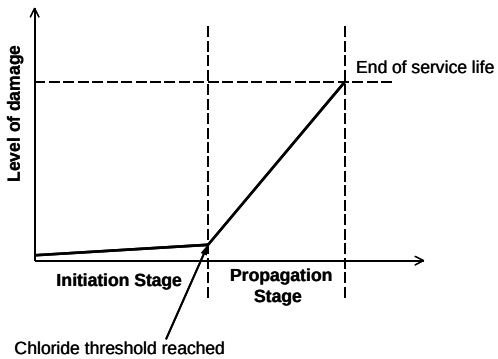

Figure 6.3 Corrosion stages (Tuutti, 1982)

There was a significant increase of the corrosion of reinforcing steels in the late 1960's and early 1970's that was attributed to clear roads policies of the 1960's which required the broad use of deicing salts. These policies allowed improved safety during poor weather conditions, but drastically changed the environmental exposure conditions of concrete highways.

Though models for chloride ingress, carbonation and corrosion development have been studied (e.g. COLLEPARDI et al., 1972, BODDY et al., 1999, BENTZ et al., 2001, ALISA et al., 1999, PAPADAKIS et al., 1992, 2000, ŠMERDA et al., 1992) including those from the probabilistic standpoint (KERŠNER et al., 1996, TEPLÝ et al., 1999, DAIGLE et al., 2004, THOFT-CHRISTENSEN, 2005), there are still many issues that must be addressed for them to become useful engineering tools, especially with regards to reliability models that can be readily used by agencies and professionals.

Diffusion is the primary mean by which chlorides penetrate to the level of reinforcing steel to initiate corrosion. The effects of hydraulic pressure and capillary absorption are minor in comparison in most cases and rarely driving factors in highway structures. It is widely accepted that Fick's 2nd law of diffusion can represent the rate of chloride penetration into concrete as a function of depth and time (Konecny et al, 2006). The solution (referred to as the Crank Solution) of the governing differential equation is given as Equation 2 (Collepardi et al., 1972)

$$C_{x,t} = C_0\left[1 - erf\left(\frac{x}{\sqrt{4D_c t}}\right)\right] \tag{1}$$

where $C_{x,t}$ is the concentration of chlorides (percent by mass of total cementitious materials) at time t (years) and depth x (meters), C_0 is the concentration of chlorides (% by mass of total cementitious materials) at the surface directly inside the concrete and D_c is the apparent diffusion coefficient (m^2/year) Equation (1) is widely used for chloride ingress models but does not account for cracks and must be modified to account for time dependent changes in material property or boundary conditions.

Severity of the chloride ingress can be assessed by comparing the chloride threshold value at which corrosion initiates, C_{th}, with the chloride concentration at the exposed areas of reinforcing steel. This value will depend on the type and preparation of the reinforcing steel and the constituents of the concrete as well as other factors. Typical values are 0.2 percent chlorides by mass of total cementitious materials according to ACI 207R-01 and 0.4 percent in the Eurocode 3 on Concrete Structures. The reliability, RF_t, of a bridge deck is expressed as the time-dependent exceedance of the corrosion threshold by the location dependent chloride concentration, $C_{xy,t}$ The reliability function characterizing the above described limit state is expressed as:

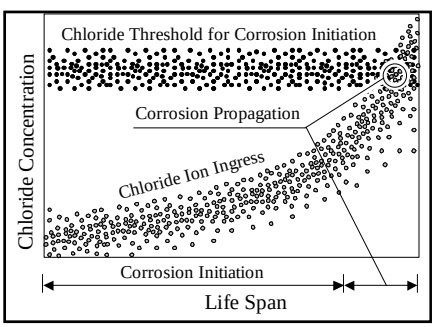

Fig. 6.4 Reliability analysis of chloride ion concentration vs. corrosion threshold

$$RF_t = C_{th} - C_{xy,t} \tag{2}$$

Probabilistic time-dependent analysis can be thought of as a comparison of the joining extrema of the chloride concentration C_t and threshold C_{th} random realizations, as show in Figure 6.4. Once the probability that the chloride concentration at the reinforcing steel level exceeds the

threshold by a user-defined amount (dependent on structure importance), corrosion is assumed to begin and the structure is designated as unreliable in terms of further delaying the onset of corrosion.

Konecny et al. (2006) provide the most advance model for understanding the ingress of chlorides and the related factors. Using simulation based reliability assessment (SBRA) with finite elements, Konecny et al. were able to show that concrete with large cracks and poor quality ECR coating have marginal service lives, but concrete with thin cracks and ECR that meets minimum quality control standards have long service lives. Figure 6.5 shows a simulation of a bridge deck with deicing salt applications.

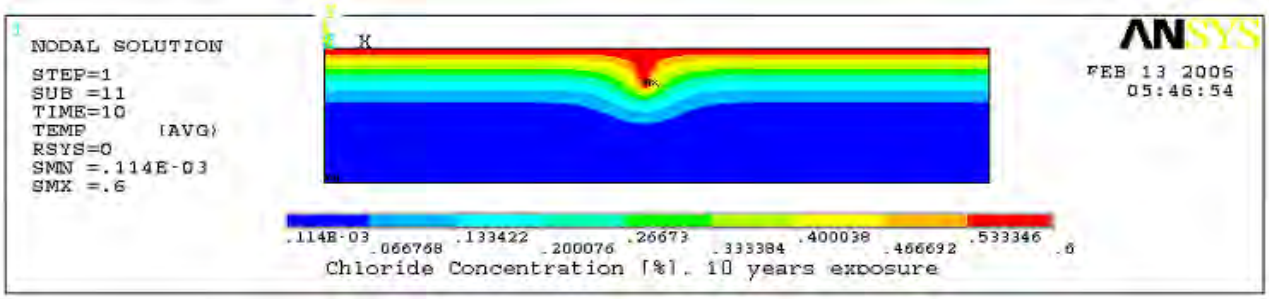

Fig. 6.5 Chloride Ion Concentration in Concrete Slab with Crack, t = 10 years

6.4 REINFORCING MATERIALS

6.4.1 Black Steel

There are many different types of reinforcement available for reinforced concrete. The cost and value of different materials is largely dependent on the design and in-service exposure conditions. The most common is “black” steel rebars. A much smaller market exists for galvanized and epoxy coated steel rebars. There are experimental bars and bars that have not been widely available until recently, including glass and carbon fiber reinforcing bars, stainless steel rebars and stainless clad rebars. The costs of these materials vary with market conditions. The April 2008 FOB costs in Salt Lake City, UT are listed in Table 6.1.

Table 6.1 Cost of Reinforcing (2008)

Type of Reinforcement	Price ($/lb) *
Black Mild Steel	0.45-0.55
Galvanized	0.55-0.65
Epoxy Coated	0.70-0.80
FRP*	0.70-0.75*
Stainless	2.50-5.00
Stainless Steel Clad	2.50-3.00
MMFX	0.70-0.80

*Price of FRP ($/ft) equivalent to a #5 bar in bulk

Black steel meeting ASTM A615 has the lowest initial cost of all the reinforcement considered. Black steel has tensile strengths usually between 60 and 75ksi and is used in applications where the structure will not be exposed to corrosive materials at a depth where the reinforcement is located. Corrosion resistance for black steel is gained by the alkaline environment creating a sacrificial coating over the steel. The passivation will likely be compromised and corrosion begins with a chloride ion concentration of approximately 0.4 percent. For mixtures that resist chloride ion penetration for the projected life of the structure, black steel may be an acceptable inexpensive alternative. However, care must be taken when specifying black steel for projects that the structure will not be exposed to corrosive environments. Many southern state DOTs specify black steel for projects. The black steel can last between 20~30 years before replacement, depending on the environment, cover, salt application, etc. With the proper cover and HPC with little cracking, black steel could potentially lead to a maximum 75-year useful life for a structure, even in Utah with heavy salt applications.

6.4.2 Epoxy Coated Steel

Epoxy coated rebar (ECR), ASTM A775, is currently the most commonly specified reinforcement by state DOTs. The FHWA and the NBS began testing organic coatings to protect the steel reinforcement from the corrosive effects of deicing salts in the 1970's. In 1973, the Pennsylvania

Figure 6.6 Epoxy coated reinforcing

Department of Transportation starting experimenting with ECR and in 1976 implemented in into all of their bridge work. Since then, nearly every DOT has adopted ECR in applications with exposure to chloride salts. The Pennsylvania and the New York DOT have not yet replaced a single bridge because of corrosion of ECR. While the ECR has certainly shown isolated areas of distress, it largely has served much longer than black bar. Premature corrosion of ECR in Sunshine Skyway Bride in Florida has created questions about the long term performance of ECR in marine environments (Hartt, Lysogorski, & Leroux, 2004), however other states have not experience the same magnitude of distress. Flexible epoxy coatings are typically colored green and can be bent or shaped after the coating has been applied. Grey, red, or purple coatings are non- flexible coatings that are to be applied after the bars have been bent or cages assembled.

Epoxy-coated reinforcement (ECR) has gained mainstream acceptance since the early 1980s as a means to extend the useful life of highway structures. The epoxy coating prevents moisture and chlorides from reaching the surface of the reinforcing steel by acting as a barrier. Research to date estimates additional service life between 40 to 50 years with plain portland cement concrete and ECR. When ECR is used in concert with HPC service lives may be expected to be 85 to 100 years. Generally, the performance of ECR has been good. However no ECR structures have been in service long enough to evaluate the true live, only estimates can be made. Bridge decks in Iowa have been reported to have gone 20 years with no signs of corrosion of the reinforcing steel (Jolley, Fanous, Phares, & Wipf, 2005) and bridges in Pennsylvania and New York more than 30 year (Camisa and Tikalsky, 2005). Most State DOTs are currently specifying ECR for roads exposed to deicing salts. Sections taken from I-15 near Salt Lake City in the late 1990's showed severe delaminations in ERC decks, however these decks were overlain with asphalt driving surfaces. Such an overlay is a design defect, as it impresses a current through the connection of two dissimilar materials, and there is a salt saturated layer in the center. This is essentially a very large battery to drive the corrosion cell.

The corrosion protection for ECR, compared with to that of black steel, is only as good as the coating. Bars need to be handled with care in order to prevent damaging the coating which would reduce the corrosion resistance of the bars greatly. Nearly all research (e.g. Humphreys, 2004; Konecny, Tikalsky and Tepke, 2007; Jolley, Fanous, Phares, & Wipf, 2005; Lee, and Krauss, 2004; Camisa and Tikalsky, 2005; Cui and Krauss, 2006) with the exception of that conducted by a group a Virginia Tech (Brown, Weyers, & Via, 2003) have found the ECR

substantially increases the life of bridges and structures. While the research indicates a gradual loss of adhesion between the bars and the coating over time the rebars do not disintegrate like those of black steel. Even the VA Tech researchers report a 12 percent increase in life with an increase to the overall cost of the bridge of less than 1 percent.

ECR has some advantages and disadvantages in its material characteristics when used in concrete structures:

- Advantages
    - Relatively inexpensive.
    - Coating that protects the steel from the corrosive environment.
    - Readily available in most areas.
    - Quality control appears to be improving.
- Disadvantages
    - Holidays can cause concentrated areas of corrosion.
    - Special handling precautions are required to avoid damaging coating.
    - Adhesion between the coating and the steel decreases over time.

6.4.3 Galvanized Steel

Hot dip galvanizing is a process that applies a zinc coating to the carbon steel rebar by immersing the bars in molten zinc (~450° C). This creates a coating consisting of an inner core of the base steel, a steel zinc alloy layer, and an outer layer of pure zinc.

Figure 6.7 Galvanized Steel

- Advantages
    - Relatively inexpensive
    - Higher threshold for initiation of corrosion
    - No special handling requirements.
    - Much greater adhesion than ECR
- Disadvantages
    - Only provides corrosion resistance until zinc is consumed
    - Special handling precautions are required to avoid damaging coating.

Galvanized steel is being used by several transportation agencies. Galvanized steel has a

sacrificial zinc coating that will corrode without expanding, however after the sacrificial layer is consumed; corrosion of the black steel core begins. The corrosion products of the zinc are not expansive; therefore they do not create the same internal stresses as the corrosion products of zinc. Galvanized steel delays the onset of corrosion by 10 or more years from that of black steel.

6.4.4 Stainless Steel

Stainless steel has long been considered cost prohibitive based on an initial cost. With more agencies considering life cycle costs, stainless steel can be a competitive alternative to traditional cost, despite the higher initial cost. Stainless steels not meeting ASTM C 955 should not be considered.

Stainless steel typically contains between 15- 30% chromium and has very high resistance to corrosion. Rapid corrosion tests performed by many different researchers consistently rank stainless steels to have the highest resistance to corrosion. Due to the high cost, this alternative has not been widely used. There are several different types of stainless steel, ASTM C 955 allows for 6 types for use in reinforced concrete. Each has different corrosion properties, as shown in Figure 6.8.

1) 2201
2) 205 typically $3.50 per pound
3) 304
4) 316
5) 316LN typically $4.50 per pound
6) 3Cr12

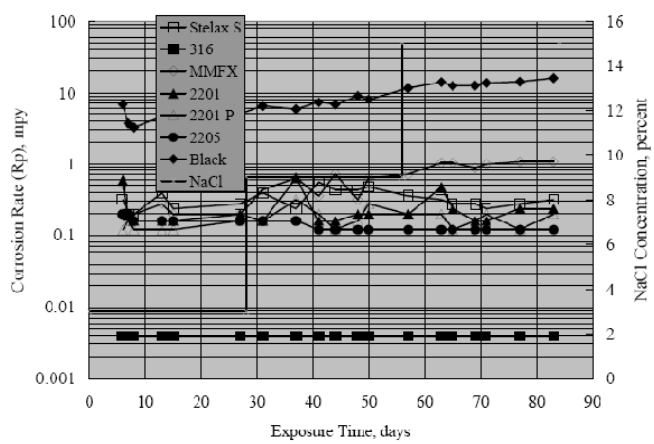

Figure 6.8 Corrosion resistance of stainless steel alloys

The stainless steel reinforcement has some advantages and disadvantages in its material characteristics when used in concrete structures:

- Advantages
    - High corrosion resistance

- Disadvantages
    - Highest cost.
    - Some concerns about ductility

A recent study concluded that the use of stainless steel (316LN) reinforcing bar is the preferred recommendation as the bridge deck corrosion protection system under the most severe exposure conditions. The use of stainless steel (316LN) reinforcing steel is also recommended for coastal substructures. That same report demonstrated that the additional cost of stainless steel reinforcement is less than the cost of a single rehabilitative overlay for a bridge deck that does not reach 75-year design life and that stainless steel reinforcement may be implemented selectively for decks subject to the most severe exposures (Brown, Weyers, & Via, 2003). Stainless 2201 and 2205 steel alloys were used in Florida (Hartt, Powers, Lysogorski, Liroux, & Virmani, Corrosion Resitant Alloys for Reinforced Concrete, 2007) with excellent corrosion resistance as measured by accelerated corrosion tests.

There are several steel mills that are trying to produce low-cost, corrosion resistant grades of stainless steel. Several new steels are available at very reasonable costs which could easily reach 100 year service life. These new steels should be investigated to ensure that the properties are appropriate for use as reinforcement in concrete structures. Enduramet32™ is a new grade of stainless steel that claims good corrosion resistance and performed well in preliminary tests. Enduramet32™ will sell for around $2.90/ lb. Arminox™ steel also has a new grade of duplex steel for around the same price. Further investigation into the properties of the new steel is recommended before specifying the material.

6.4.5 Stainless Steel Clad

Stainless steel clad rebar is currently produced by two known processes. In one of the processes, stainless steel strip is formed and welded into a tube shape. Carbon steel granulate is then packed under pressure into the tube to form the core. The ends are crimped to complete the “manufactured” round billet. The billet is then heated and rolled into reinforcing bars, as shown in Figure 6.9. In the other existing process, a carbon steel continuous cast billet is spray metallized with a stainless alloy cladding. Then the billet is heated and rolled into reinforcing bars.

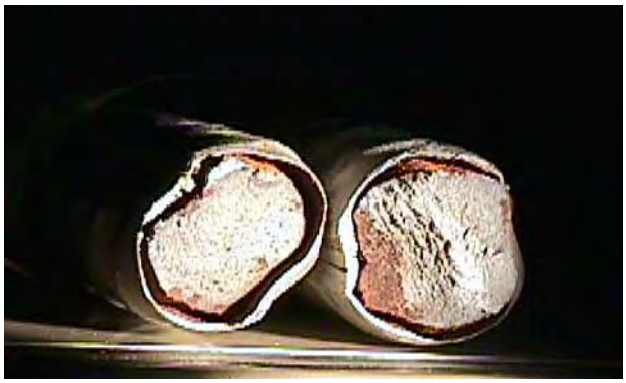

Figure 6.9 Stainless steel clad reinforcing

Several DOTs have used SSC experimentally, e.g. Kentucky, South Carolina, Virginia, West Virginia, Wisconsin, Oregon, Florida, and South Dakota. The SSC rebar estimated to give 50-60 years of life before damaging the concrete. Abrading the cladding reduced the life estimate by a few years, usually 1-5 years. Drilling a hole in the cladding, to simulate a break, significantly reduced the estimated life of the end coated SSC rebar by 15-40 years (Cross, Duke, Kellar, Han, & Johnston, 2001; Clementa, 2004). The SSC rebar with end coating is estimated to give 50-60 years of life before damaging the concrete.

The SSC reinforcement has some advantages and disadvantages in its material characteristics when used in concrete structures:

- Advantages
    - Corrosion resistant layer.
    - Less expensive than stainless but similar properties.
    - High life expectancy under ideal material properties
- Disadvantages
    - High cost.
    - Non uniform thickness of cladding.
    - Defects can cause concentrated areas of corrosion. Carbon steel is less noble than stainless and therefore will corrode in preference to the stainless steel.
    - Different metals with slightly different coefficients of expansion.
    - Gaps between the cladding and the core.
    - Supply may not meet demand.
    - Requires special treatment of the exposed ends.

There have been reports of potential problems with the uniformity of the thickness of the

cladding. Studies conducted by South Dakota DOT, Florida DOT, and Oregon DOT found that the yield strength may actually be less than required by specs for ¾" bars. There are also production limitations restricting smaller diameter bars. Separation of the cladding from the core is also a major concern. There have also been reports of significant delays in delivery schedules.

Despite all of the disadvantages of the product, there are significant benefits of the product if proper manufacturing and construction measures are taken.

Grades of different stainless steels and the corrosion resistance versus exposure times are shown below in Figure 6.10.

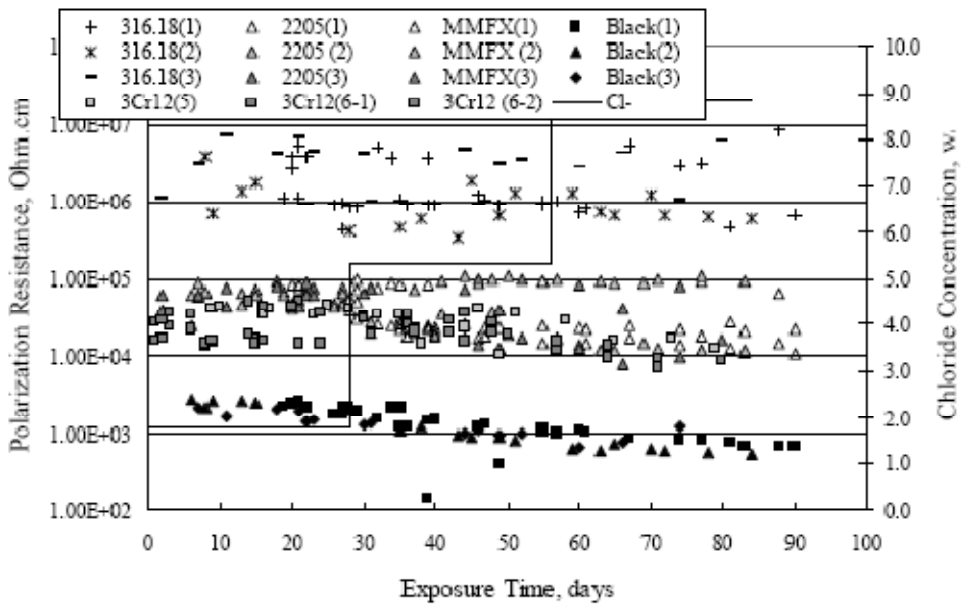

Figure 6.10 Polarization resistance versus exposure time for different alloys (Hartt et al., 2007)

6.4.6 Fiber Reinforced Polymer

In recent efforts to solve the corrosion problems in concrete, nonmetallic materials such as fiber reinforced polymer (FRP) composites have become an alternative to reinforcing steel in various concrete structures. FRP reinforcement is primarily made of fibers embedded in a thermosetting polymer or thermoplastic resin. The small diameter inorganic and organic fibers (e.g., glass, carbon, aramid, and polyvinyl alcohol) provide FRP reinforcement with strength and stiffness, whereas the polymer resins (e.g., polyester, vinyl ester, and epoxy) bind the fibers

together. In addition, inorganic fillers (e.g., calcium carbonate, clay, and alumina trihydrate) can be mixed with the resins for cost reduction, property modification, and processing property control of FRP reinforcement.

The FRP reinforcement has some advantages and disadvantages in its material characteristics when used in concrete structures:

- Advantages
    - High longitudinal strength.
    - Nonmagnetic.
    - Corrosion resistance.
    - High fatigue endurance.
    - Light weight.
    - Reduced lap splices because of the availability of 40’ length bars
    - Low thermal and electric conductivity.
- Disadvantages
    - Very little yielding before brittle rupture.
    - Low transverse strength.
    - Low modulus of elasticity.
    - Susceptibility to damage due to ultra-violet radiation.
    - Low durability of some glass fibers in a moist environment.
    - Low durability of some glass and aramid fibers in an alkaline environment.

The material characteristics of FRP need to be carefully considered when determining whether FRP reinforcement is suitable or necessary for a particular concrete structure. There are several commercially available FRP reinforcements made of continuous aramid (AFRP), carbon (CFRP) or glass (GFRP) fibers embedded in various resin materials. Also, FRP reinforcements can be sorted by the type of surface deformation system, such as exterior wound fibers, sand coating, or separately formed deformation (Nanni & Faza, Designing and Constructing with FRP Bars: An Emerging Technology, 2002). The price of FRP bars has decreased dramatically and now can be competitive with ECR. The construction of an FRP reinforced bridge deck may actually cost less than the same deck reinforced with ECR. Wisconsin DOT reported a 57%

savings in man hours required to place the pars because of the low weight of the material (Berg, Bank, Oliva, & Russell, 2006). There may also be an initial savings cost associated with using a lighter material and thereby reducing the dead loads and girder size. There is also less of a demand for impervious concrete and strict curing procedures so there could be a reduction in cost associated with that. Also many manufacturers can supply the bars in 40' lengths thereby reducing the lap splices and saving additional materials.

6.4.7 MMFX Steel

MMFX steel is a low carbon steel containing about 9% chromium (ASTM 1065). The technology was developed at UC Berkley roughly 10 years ago. Reports from the MMFX technologies and various DOTs and Universities rate the corrosion resistance from moderate to excellent. MMFX did not perform well in a salt fog study (Darwin, Browning, Nguyen, & Locke, 2002) and the Florida DOT has not completed its evaluation. Because MMFX is a relatively new technology no long term performance data exists. MMFX has a tensile strength of 100 ksi and an ultimate strength of 120 ksi.

The MMFX reinforcement has some advantages and disadvantages in its material characteristics when used in concrete structures:

- Advantages
    - Relatively low cost.
    - High strength.
    - Corrosion resistance.
    - Ductile.
- Disadvantages
    - Only one supplier.
    - No specifications exist.
    - No long term data exists.
    - Corrosion resistance may not be sufficient.

Figure 6.11 MMFX Reinforcing

Predicted useful life of RC structures using MMFX steel vary from 55- 100+ years. These predictions are based on accelerated corrosion tests. A study conducted by the Kansas DOT in conjunction with South Dakota DOT reported that ECR actually performed better in

Accelerated Corrosion Tests (ACT) than MMFX. Figure 5 shown below displays some of the results of the study. Most other studies conclude that the corrosion resistance of MMFX is equivalent or better than ECR.

There is only one supplier for the product and supply and cost have been concerns. The research to date indicates a corrosion resistance typically four to eight times that of uncoated reinforcement, and a one-third to two-thirds lower corrosion rate. That translates to an initial bridge deck service life estimate of 52 years before repairs are needed. Life cycle cost analysis over a 90- year analysis period indicated a $31/yd^2 lower cost of MMFX compared to ECR.

Studies done by Florida DOT found ductile behavior of MMFX was the same as grade 60 mild steel, but lap splices were not. Study commissioned by MMFX found that the Young's modulus varied with stress level and was limited to 40 ksi based on limiting crack width of 0.016" and deflection of L/360.

6.4.8 Fiber Reinforcement (ACI 544)

Fiber reinforcement in conjunction with bar reinforcement has the potential to greatly reduce cracking. Fiber reinforcement has existed for thousands of years as a method to control cracking in brittle materials. There are four different types of fiber reinforcing available for reinforcing concrete; steel, natural, glass, and synthetic.

Steel fiber reinforced concrete should not be used if the steel is susceptible to corrosion over the projected service life of the structure. The fiber manufacturer should be consulted in order to determine the maximum length of fiber that can be used while avoiding "balling" of the fibers. There are different fiber geometries all with different mechanical bond properties. ASTM A 820 describes the minimum tensile strength and other physical properties for steel fibers. Natural fibers should not normally be used because most natural fibers tend to swell in the presence of moisture. Also, the naturally occurring glucose in the natural fibers will retard the setting time.

Glass fiber reinforcement has traditionally been used for thin architectural precast panels. The silica in the glass should be resistant to alkali silica reaction (AR Glass) in order to prevent premature deterioration. Glass fiber reinforcement is not recommended for transportation applications. Synthetic fiber has probably the greatest potential for transportation applications.

There are currently several types of fibers available; acrylic, aramid, carbon, nylon, polyester, polyethylene, polypropylene, etc. The suitability of different fibers for different applications should be confirmed through independent third-party testing. 0.3 ~1.0 percent by volume replacements of will distribute the cracks better, decrease the width of the cracks, increase toughness, increase early age tensile strength, and decrease shrinkage. Proper equipment and dosage should be used in order to prevent “balling” of the fibers.

When specifying a material to use for reinforcement, knowledge of the use of the structure, the environment, and the design service life is critical. During accelerated corrosion tests, most reinforcement exhibited some corrosion. Corrosion resistant reinforcement along with less permeable concrete and other measures can produce durable structures with a 100 year service life or better. Life cycle cost analysis is the preferred method for choosing appropriate alternatives and indirect costs should also be considered. As use for some of these corrosion resistant reinforcement increases, the cost is expected to decrease and the availability to increase.

CHAPTER 7. CONSTRUCTION

7.1 INTRODUCTION

Durable concrete is produced by using quality materials with optimized concrete mixture designs and carefully controlled construction practices—proportioning, batching, mixing, placing, consolidating, finishing and curing. All of these are essential to produce corrosion resistant long-life structures. The contractor cannot use the standard practices of yesterday in order to produce the long-life highway structures of tomorrow. Quality control and an acute attention to detail are essential in producing a quality product. There is also a necessity for key field personnel to have an understanding of the process that will lead to durable concrete and cooperation of the crew.

7.2 PRODUCING, PLACING, FINISHING AND CURING CONCRETE

Prior to placing any concrete, engineers, contractors and owners should meet to discuss the logistics of the placement. Final plans, changes, and specifications should be checked and discussed. Any potential problems or anticipated delays along with contingency plans should be reviewed and discussed. By this time, the ready mix plant will have verified that the mixture meets the HPC specifications set forth by the engineers or agency. The contractor will also conduct a trial placement to demonstrate that he/she is qualified and competent to place the HPC concrete.

Just prior to placing the concrete, the sight should be inspected to ensure that reinforcement and formwork are in the proper placement and free of dirt and debris. The subgrade should be damp, but with no standing water. Temperatures of the formwork and concrete should be within the tolerances set forth by the engineers, contractors, and the ready mix plant. Concrete placed in hot weather can be cooled by adding ice or cold water to the mixture (American Concrete Institute, 304, 2007). Forms can be heated for late season placements in order to help with strength development and to ensure that the mixing water does not freeze. Strict QA/QC measures should be followed throughout the placement to ensure that the concrete meets specifications and those proper procedures are followed.

The ready mix plant must monitor the quantity of each specified constituent and document that the constituents are within the specified tolerances. Concrete should be rejected if

it has been in the truck for longer than 90 minutes or the materials are out of compliance. Failure to implement this basic requirement will likely lead to expensive repairs in the future. Concrete should not be discharged from a pump or a truck with more than a four foot free fall to prevent segregation (Tepke and Tikalsky, 2007). Concrete should also be discharged as close the final position as possible as lateral movement tends to cause segregation. It is also important that the concrete delivered to the site is of the same quality and that the ingredients are the same. Concrete should be vibrated to ensure proper consolidation. ACI 309R reports that internal vibrators are the most effective method of consolidating fresh concrete and that the effectiveness of the vibrator is dependent on the frequency, head diameter and amplitude.

After the concrete is properly consolidated, the concrete is finished by screeding, floating and troweling with wood or magnesium blades. The surface may be broom finished or textured as desired. The concrete should be manipulated as little as possible. Overworking the concrete will bring fines and water to the surface which will lead to scaling and surface defects. Immediately after finishing a curing regime should be undertaken. Properly cured concrete is concrete that has sufficient water for the rate of hydration. The reaction between the cementitious material and the water is an exothermic reaction and the rate is greatly influenced by the temperature. For this reason, great care should taken for hot and cold weather placement of concrete ACI 305 and ACI 306 from *The Manual of Concrete Practice* are a great resource for such placements.

Wind and evaporation rates should also be monitored to prevent drying shrinkage cracking. Curing compounds can be used in moderate weather to prevent evaporation, but are largely inadequate in hot or dry climates. Both of these are evident in Utah. Bleed rates of the concrete should not be less than the evaporation rates of the local conditions. Mixtures with low bleed rates are very vulnerable to surface drying and cracking. Such mixtures should be protected by fogging, wind screens, sun shades and/or evaporation reducers. Excess water from fogging or evaporation reducers should not be manipulated into the surface of the concrete.

Concrete should be protected from evaporation immediately after finishing by cotton matts and/or wet burlap, and plastic sheets. The duration for which moist curing should be applied is greatly dependant on the weather and design strength. Curing measures should be applied until the mixture has reached the desired properties or that it will continue to develop toward the desired properties at a rate which corresponds to the time set forth by the engineer.

Avoiding restrained shrinkage by considering the type and timing of the connection of bridge decks to rigid structural elements such as abutments, bents, and diaphragms can greatly reduce the cracking. Expansion joints can also relieve some cracking caused by restrained shrinkage. Creep, temperature, and shrinkage should all be considered when determining expansion joint. For continuous span bridges, positive moment areas should be poured and cured first. These practices will help reduce cracking in the bridge deck.

CHAPTER 8. SUMMARY

8.1 SUMMARY

The causes, variables and mitigating factors that affect corrosion of reinforcing steel in the State of Utah's transportation infrastructure have been synthesized in this report. Corrosion mitigation strategies using several different approaches are clearly options for the Utah Department of Transportation. The life-cycle approach to designing long lasting structures favors two major options.

1) The creation of a performance based specification that emphasizes low permeability, moderate shrinkage and quality control/assurance in the construction of all reinforced concrete elements. This form of HPC in combination with existing epoxy coated reinforcement can be used to create 100-year highways. This term would indicate that statistically highway structures in Utah would have a service life of 100-years. Such design and constructed structures would not typically require any form of major maintenance for more that 50-years and most not for 75-years under design conditions.
2) The State of Utah could lead an effort to eliminate major corrosion issues by eliminating steel from non-ductile structural members (bridge decks, footings, mass piers, bridge barriers and compression members. Such an effort would require the conversion of state specifications to fiber reinforced polymers reinforcing bars and textiles. This could also be done in conjunction with HPC specifications to develop an infrastructure that would serve the State of Utah for 200 or more years.

Both of these options provide multiple protection systems to the reinforcing bars in the structure. They provide concrete with low cracking potential and can be combined with improved engineering practice to minimize other forms of deterioration from dictating the life of the structure. The technologies for these measures exist today and neither option is cost prohibitive. In fact, option 1 may cost less than existing practices and last twice as long. Option 2 may slightly increase the initial capitol costs of highway structures, but would alleviate future maintenance costs and have a long life horizon.

8.2 SPECIFICATION IMPROVEMENTS

There are some immediate improvements in the existing specifications of the State of Utah Department of Transportation that would improve the durability of highway structures. These suggestions are made as “track changes” in the following pages. Comments are inserted where explanations are needed. The suggested changes will inhibit the intrusion of chlorides through both diffusion and cracks.

A specification addition to allow for FRP reinforcing would require a new section in the specification. It would follow the form of Section 03211 and would be referenced in Sections 02645, 02646, 02844, 02861, and 03339. The acceptable standard would require a thorough review of existing manufacturers products, AASHTO and ASTM discussions on creation of a standard and major effort to define where such products can be safely used. For example, bridge decks are non-ductile structural elements that typically fail from corrosion. This would be an excellent use of FRP bars. However, elements that are fatigue rated or require ductile failures may not be suitable without new design methodologies.

APPENDIX A

SECTION 03055

PORTLAND CEMENT CONCRETE

PART 1 GENERAL

1.1 SECTION INCLUDES

A. Materials and procedures for producing portland cement concrete.

1.2 RELATED SECTIONS Not Used

1.3 REFERENCES

A. AASHTO M 6: Standard Specification for Fine Aggregate for Portland Cement Concrete

B. AASHTO M 80: Standard Specification for Coarse Aggregate for Portland Cement Concrete

C. AASHTO M 154: Standard Specification for Air-Entraining Admixtures for Concrete

D. AASHTO M 157: Standard Specification for Ready-Mixed Concrete

E. AASHTO M 194: Standard Specification for Chemical Admixtures for Concrete

F. AASHTO M 295: Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete

G. ASTM C 150: Standard Specification for Portland Cement

H. ASTM C 595: Standard Specification for Blended Hydraulic Cements

I. ASTM C 1157: Standard Performance Specification for Hydraulic Cement

J. ASTM C 1240: Standard Specification for Silica Fume for Used in Cementitious Mixtures

K. ASTM C 1567: Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)

L. ASTM C 1602: Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete

M. American Concrete Institute (ACI) Standards

N. Precast/Prestressed Concrete Institute (PCI)

O. UDOT Materials Manual of Instruction

P. UDOT Minimum Sampling and Testing Requirements Manual

Q. UDOT Quality Management Plan

1.4 DEFINITIONS Not Used

1.5 SUBMITTALS

A. Furnish to the Engineer a mixture design for each class of concrete to be used.

1. Base concrete mixture designs for all "A" concrete classes on trial batch test results or on UDOT's past project history using the same materials used in previous mixture designs within the past year.
2. Use the same components in the trial batches that are to be used in the project including coarse and fine aggregate, water, source and type of all cementitious materials, air-entraining agent, chemical admixtures including any site-added admixtures intended to be used.
3. Do not exceed 50 percent total pozzolan in any mixture unless otherwise specified.
4. The Department or its representative witnesses the trial batch.
5. Mix concrete trial batches as specified in UDOT Materials Manual of Instruction Part 8-974: Guidelines for Portland Cement Concrete Mixture Design.
6. Meet the following additional requirements for Self Consolidating Mixtures (SCC):
   a. Design and mix according to ACI Manual of Concrete Practice 301: Specifications for Concrete.
   b. Provide mixture specific flow and spread criteria.
   c. Meet PCI – TR-6-03. A visual stability index rating of 0 – 1 is required.
   d. Provide compressive strength (ASTM C39), shrinkage (ASTM C157) and data.
   e. Include documentation justifying any deviation from the aggregate operating bands required by Table 4 with the mixture design for approval. Production may not begin until the deviation is approved.

B. Test results verifying the coarse and fine aggregate used meets this section, article 2.2

C. Verification that cement used is from a pre-qualified supplier. See this Section,

article 2.1, paragraph E.

D. Verification that fly ash used in from a pre-qualified supplier. See this Section, article 2.5, paragraph A.1.d.

E. Verification that the batch plant meets the requirements of the UDOT Quality Management Plan for Ready-Mix Concrete.

1.6 ACCEPTANCE

A. Acceptance is in accordance with UDOT Minimum Sampling and Testing Requirements.

B. When concrete is below specified strength and does not have a separate strength pay factor:

1. Department may accept item at a reduced price.
2. The pay factor will be applied to the portion of the item that is represented by the strength tests that fall below specified strength.
3. Department will calculate the pay factor as follows based on 28 day compressive strength:

Psi below specified strength:	**Pay Factor:**
1 – 100	0.95
101 – 200	0.90
201 – 300	0.85
301 – 400	0.80
More than 400	0.50 or Engineer may reject

PART 2 PRODUCTS

2.1 CONCRETE CLASSES AND MIXTURE REQUIREMENTS

A. Meet the requirements in Table 1.

Table 1

Concrete Classes and Mix Requirements							
Class	**Nominal Maximum Coarse Aggregate Size**	**Max. Water/ Cementitious Ratio**	**Min. Cementitious Content (lb/yd^3)**	**Slump (Inch) See Article G for further Criteria**	**Air Content Percent (%)***	**Mixture Design Compressive *f*'cr (psi)****	**28 Day Minimum Compressive *f*'c (psi) *****
AA(AE)	2" to 1-½" 1" ¾"	0.44 0.44	564 564	3.0 to 5.5 . 3.0 to 5.5	4.0 - 7.0 5.0 - 7.5	5200 5200	4000 4000
A(AE)	1-½" to No. 4 1" to No. 4 ¾" to No. 4	0.53 0.53 0.48	470 470 517	1 to 3.5 1 to 3.5 1 to 3.5	4.5 - 7.5 4.5 - 7.5 4.5 - 7.5	3900 3900 3900	3000 3000 3000
B or B(AE)		0.62	376	2 to 5	-- 3.0 - 6.0	3250	2500

* Values listed represent in-place air content. Make necessary adjustments for impacts to air content due to placement.

** ***f*'cr may be based on statistical analysis of established mixture to meet *f*'cr> *f*'c+1.34σ and *f*'cr>*f*'c+2.33σ-500**

*** For *f*'c over 4000 psi, design and proportion mixtures according to ACI 301: Specifications for Concrete and project specific criteria.

B. Minimum strength is based on a coefficient of variation of 10 percent, and one test below the minimum strength per 100 tests.

C. Maximum nominal size of coarse aggregate:

1. Not larger than $^1/_5$ of the narrowest dimension between sides of forms.
2. Not larger than ⅓ the depth of slabs.
3. Not larger than ¾ of the minimum clear distance between reinforcing bars or between bars and forms, whichever is less.

D. Do not exceed water/cementitious ratio.

E. Calculate the water/cementitious ratio (w/c) according to the following formula:

$$\frac{W}{C} = \frac{Water}{Cement + Pozzolan}$$

F. Use 94 lb more cement per cubic yard when concrete is deposited in water than the design requires for concrete placed above water.

G. Use Table 4 to determine the slump requirements when not using water-reducing admixtures or viscosity modifying admixtures.

1. Slump requirements when using low range water reducers: 1 inch to 5 inches for all classes of concrete.
2. Slump requirements when using high range water reducers: 4 inches to 9 inches for all classes of concrete.
3. Slump requirements when using viscosity modifying admixtures: None. Meet visual stability index of 0 – 1.

2.2 CEMENT

A. Use type II portland cement or blended hydraulic cement equivalents from Table 2 unless otherwise specified. (ASTM C 150, ASTM C 595, ASTM C 1157)

B. Portland Cement

1. Follow Tables 1 and 3 in ASTM C 150.
2. Follow the requirements of Table 2 of ASTM C 150 for low-alkali cement.

C. Blended Hydraulic Cement.

1. When blended hydraulic cement is substituted for portland cement:
   a. Use ASTM C 1567 to verify that expansion is less than 0.1 percent at 16 days.
   b. Refer to the equivalent cements listed in Table 2.
2. Do not exceed 35 percent total pozzolan limit when adding flyash to a blended hydraulic cement.
   a. Submit documentation of the total pozzolan content with the mixture design.

Table 2

Portland Cement/Blended Hydraulic Cement Equivalencies		
ASTM C 150 (Low Alkali)	**ASTM C 595**	**ASTM C 1157**
Type I	IP	GU
Type II	IP (MS)	MS
Type III	-	HE
Type V	-	HS

D. Do not use cement that contains lumps or is partially set.

E. Use cement from the list of UDOT qualified suppliers list maintained by the UDOT Materials Quality Assurance Section.

F. Do not mix cement originating from different sources.

G. Department will sample and test the cement in accordance with UDOT Quality Management Plan 502: Cement.

2.3 AGGREGATE

A. Coarse Aggregate for Normal Concrete Mixtures

1. Use coarse aggregate meeting AASHTO M 80 physical properties. Use one of the gradations found in Table 2.
2. Do not exceed 1 percent of deleterious substances as shown in AASHTO M 80, Table 2, for Class A aggregates. Material finer than No. 200 sieve: maximum allowable 1 percent, exception as noted in footnote d.

B. Fine Aggregate for Normal Concrete Mixes

1. Use fine aggregate meeting AASHTO M 6 physical properties. Use the gradation found in Table 3.
2. Do not exceed 3.0 percent of deleterious substances as outlined in AASHTO M 6, Table 2, for Class A aggregates, using option “b” for material finer than the No. 200 sieve. Material finer than No. 200 sieve: maximum allowable 3 percent.

Table 3

Aggregate Gradations - Percent Passing (by weight)								
Aggregate or Sieve Size (inches)		**2**	**1½**	**1**	**¾**	**½**	**⅜**	**No. 4**
2 to No. 4		95-100	80-90	35-70		10-30		0-5
1½ to No. 4			95-100	80-90	35-70		10-30	0-5
1 to No. 4				95-100	70-90	25-60		0-10
¾ to No. 4				100	90-100	60-85	20-55	0-10

Table 4

Gradation	
Sieve Size	**Percent Passing (by weight)**
⅜ inch	100
No. 4	95 to 100
No. 8	70-90
No. 16	45 to 80
No. 30	25-60
No. 50	5 to 30
No. 100	0 to 10

C. Coarse and Fine Aggregate for Self Consolidating Concrete (SCC) Mixtures.

1. Combined gradations of coarse and fine aggregates must be within the bands shown in Table 4. Establish targets and production tolerances necessary to meet the requirements of Table 4.

Table 5

Aggregate Gradations (Percent Passing by Dry Weight of Aggregate)		
Sieve Size	**¾ inch Operating Bands**	**½ inch Operating Bands**
¾ inch	95 – 100	–
½ inch	65 – 95	95 –100
⅜ inch	58 – 83	65 – 95
No. 4	35 – 65	50 – 80
No. 8	25 – 50	30 – 60
No. 16	15 – 35	20 – 45
No. 30	10 – 35	12 –35
No. 50	5 – 20	5 – 20
No. 100	1 – 12	2 – 12
No. 200	0 – 2	0 – 2

2.4 WATER

A. Use potable water or water meeting ASTM C 1602, including Table 2.

B. Screen out extraneous material when pumping water from streams, ponds, lakes, etc.

2.5 ADMIXTURES

A. Air Entrainment: as specified. Meet AASHTO M 154, including Section 5.

B. Water Reducing Agents: Meet AASHTO M 194.

1. High Range Water Reducer (HRWR): Submit a written plan for approval with the trial batch that shows proper attention will be given to ingredients, production methods, handling and placing.
2. Do not use calcium chloride.

C. Accelerators: Meet AASHTO M 194

1. Use non-chloride accelerators.

D. Set Retarding Admixtures: Meet AASHTO M 194.

1. Establish the effective life of the set-retarding admixture by trial batch if set retarding admixtures are required due to haul times exceeding the time limitations in this Section, article 3.4, paragraph A.
2. Do not exceed any manufacturer recommendations for the use of the set-retarding admixture.
3. Do not re-dose the concrete with additional set retarding admixture.
4. Add set retarding admixture at the batch plant at the time of initial batching operations.
5. Show on batch tickets the amount of admixture used.
6. Time of placement is established by the trial batch and supersedes the requirements in this Section, article 3.4, paragraph A.

E. Viscosity Modifying Admixtures.

1. Do not exceed any manufacturer recommendations for the use of the viscosity modifying admixture.
2. Do not re-dose the concrete with additional viscosity modifying admixture.
3. Show on batch tickets the amount of admixture used.

F. Site-added admixtures.

1. Use admixture in the trial batch as site-added admixtures only with the written concent of the agency.
2. Use pre-measured admixtures only.
3. Record amount used on batch ticket.
4. Rotate the drum at least 30 revolutions at the mixing speed recommended by the manufacturer.

2.6 POZZOLAN

A. Fly Ash:

1. Class F, as specified. Conform to AASHTO M 295 except table 2.

a. Replace a minimum of 20 percent of the portland cement by weight unless otherwise specified. Use the minimum cement content in the design formulas before replacement is made.

b. Loss on Ignition (LOI): not to exceed 3 percent.

c. Maximum allowable CaO content: not to exceed 15 percent.

d. Use fly ash from the list of UDOT pre-qualified sources maintained by the UDOT Materials Quality Assurance.

e. Label the storage silo for fly ash to distinguish it from cement.

f. Use different size unloading hoses and fittings for cement and fly ash.

2. Fly ash may be sampled and tested for compliance at any time.

B. Natural Pozzolan (Class N)

1. Conform to AASHTO M 295.

2. May use instead of fly ash provided that the expansion, according to ASTM C 1567, does not exceed 0.1 percent.

C. Silica Fume: Conform to ASTM C 1240.

D. Slag Cement:

1. Conform to ASTM C989.

2. Replace a minimum of 35 percent of the portland cement by weight unless otherwise specified. Use the minimum cement content in the design formulas before replacement is made.

PART 3 EXECUTION

3.1 PREPARATION

A. Aggregate stockpiles:

1. Construct stockpile platforms so that subgrades are prevented from intruding into aggregates.

2. Build stockpiles at least two days before use.

3. Provide an operator and front-end loader to help the Engineer take

aggregate samples.

4. Aggregate may be accepted in daily increments, but not more than 30 days before use.
5. Provide separate stockpiles for coarse and fine aggregate.
6. Construct stockpiles to minimize segregation of aggregate
7. Allow washed aggregates to drain to uniform moisture content before use (12 hours minimum).

3.2 BATCH MATERIALS

A. Meet AASHTO M 157.

B. Hand Mixing:
   1. Only Class B concrete may be hand mixed.
   2. Hand-mixed batches cannot exceed 0.5 yd^3.
   3. Hand mix on a watertight platform.
   4. Spread the aggregate evenly on the platform and thoroughly mix in the dry cement until the mixture becomes uniform in color.

C. Truck-Mixed Concrete (Dry-Batch):
   1. Do not load trucks in excess of their rated mixing capacity, or 63 percent of the drum gross volume, or less than 2 yd^3.
   2. The truck rating plate must be readable.

3.3 MIXTURE DESIGN

A. Do not place concrete without written approval of the mixture design.

B. Do not change the mixture design without written approval.

3.4 LIMITATIONS – GENERAL

A. Timing. Unless otherwise specified, place concrete:

1. Within 90 minutes of batching when the air temperature is below 80 degrees F.
2. Within 75 minutes of batching when the air temperature is between 80 and 85 degrees F.
3. Within 60 minutes of batching when the air temperature is between 86 and 90 degrees F.
4. Prior to initial set.

B. Concrete Temperature: Unless otherwise specified, place concrete in the forms when the concrete temperature is between 50 and 80 degrees F.

C. Pumping and Conveying Equipment

1. Do not use equipment or a combination of equipment and the configuration of that equipment that causes a loss of entrained air content that exceeds one half of the range of air content allowed by specification.
2. Contractor is responsible for verification and monitoring of air loss.

3.5 CYLINDER STORAGE DEVICE

A. Provide and maintain cylinder storage device.

1. Maintain cylinders at a temperature range of 60 degrees F to 80 degrees F for the initial 16-hour curing period.
2. Do not move the cylinders during this period.
3. Equip the storage device with an automatic 24-hour temperature recorder that continuously records on a time-temperature chart with an accuracy of ±1 degree F.
4. Have the storage device available at the point of placement at least 24 hours before placement.
5. Engineer stops placement of concrete if the storage device cannot accommodate the required number of test cylinders.
6. Use water containing hydrated lime if water is to be in contact with cylinders.
7. A 24-hour test run may be required.

APPENDIX B
SECTION 03310

STRUCTURAL CONCRETE

PART 1 GENERAL

1.1 SECTION INCLUDES

A. Materials and procedures for constructing structural concrete, including box culverts, concrete slope protection, diversion boxes, catch basins, cleanout boxes and other items as specified.

B. High Early Strength Concrete for closure joint at each end of bridge deck and the longitudinal or transverse closure joints between all the precast concrete deck panels, bridge parapets and approach slabs as shown on plans.

1.2 RELATED SECTIONS

A. Section 00555: Prosecution and Progress

B. Section 02317: Structural Excavation

C. Section 02752: Portland Cement Concrete Pavement

D. Section 03055: Portland Cement Concrete

E. Section 03152: Concrete Joint Control

F. Section 03211: Reinforcing Steel and Welded Wire

G. Section 03390: Concrete Curing

H. Section 05822: Bearings

I. Section 05832: Expansion Joints

1.3 REFERENCES

A. AASHTO M 85: Standard Specification for Portland Cement (Chemical and Physical)

B. AASHTO M 111: Zinc (Hot-dip Galvanized) Coatings on Iron and Steel Products

C. AASHTO M 148: Liquid Membrane-Forming Compounds for Curing Concrete

D. AASHTO M 153: Preformed Sponge Rubber and Cork Expansion Joint Fillers for Concrete Paving and Structural Construction

E. AASHTO M 213: Preformed Expansion Joint Fillers for Concrete Paving and Structural Construction (Nonextruding and Resilient Bituminous Types)

F. AASHTO M 235: Epoxy Resin Adhesives

G. AASHTO LRFD Bridge Construction Specifications Section 3 (Temporary Works)

H. ASTM C 578: Rigid, Cellular Polystyrene Thermal Insulation

I. American Concrete Institute (ACI) Standards

1.4 DEFINITIONS Not Used

1.5 SUBMITTALS

A. Falsework Drawing:

1. Submit for approval at least one week before construction starts, three copies of falsework drawings and design calculations signed and sealed by a professional engineer licensed in the State of Utah when required in the contract or requested by the Engineer.
2. Comply with AASHTO LRFD Bridge Construction Specifications Section

3 (Temporary Works).

B. Design and submit to the Engineer for approval when specified in the plans, a High Early Strength Concrete mix design, which attains a 24 hour compressive strength of 3000 psi and a 28-day compressive strength f_c of 4000 psi minimum. Provide a certificate stating that the mix submitted meets the requirements for coarse aggregate, fine aggregate, cement, water, admixtures and curing materials in Section 03055 at least two weeks before its use.

C. Cold Weather Plan according to this Section, article 3.8.

D. Surface Evaporation Plan according to this Section, article 3.8.

1.6 ACCEPTANCE – Price Adjustments for Strength and Performance Criteria

A. Use a pay factor of 1.0 when concrete strength and/or performance measures meet or exceeds the specified strength and performance criterion.

B. When concrete is below specified strength or performance measures:

1. Department may accept item at a reduced price.
2. The pay factor applies to the portion of the item that is represented by the strength tests that fall below specified strength or the performance test that falls below the performance specification.
3. Department calculates the pay factor as follows:

Percent below specified strength:	**Pay Factor:**
0-2 percent	0.9
2-4 percent	0.8
4-6 percent	0.7
6-8 percent	0.6
8-10 percent	0.5

4. Remove and replace all concrete represented by the test if the concrete strength or performance measure is less than 90 percent of the specified strength or performance criterion.

PART 2 PRODUCTS

2.1 CONCRETE

A. Class AA(AE) concrete, unless specified otherwise.

1. Meet a 28-day compressive strength of 4000 psi, drying shrinkage less than 600 microstrain (ASTM C157) and chloride ion resistance (AASHTO T277) less than 2500 or as specified by the agency. Values will be verified through trial batch.

B. Concrete Slope Protection: Class A(AE).

C. Refer to Section 03055.

D. For High Early Strength Concrete use air-entrained concrete meeting Section 03055.

1. Use either air-entraining portland cement or an approved air-entraining admixture to obtain the air-entraining feature.

a. The entrained air content shall conform to Section 03055

2. Conform to the requirements of AASHTO M 85, ASTM C595 or ASTM C1157 for portland cement and blended cement.

2.2 REINFORCING STEEL AND WELDED WIRE

A. Refer to Section 03211.

2.3 JOINTS AND SEALERS

A. Pre-Molded Joint Filler meeting AASHTO M 153.

1. Concrete Slope Protection: Refer to Section 03152.

B. Preformed Joint Filler: AASHTO M 213.

2.4 BACKER ROD

A. Use backer rod composed of closed-cell polyethylene foam of sufficient size to prevent the sealant from passing to the bottom of the groove.

B. Refer to Section 03152.

2.5 WATERSTOPS

A. Refer to Section 03152.

2.6 RIGID PLASTIC FOAM

A. Preformed, extruded, cellular polystyrene thermal insulation material that has a water absorption property of 0.3 or less.

B. Refer to ASTM C 578.

2.7 CURING COMPOUND

A. As specified. AASHTO M 148, Type I-D, Class A.

2.8 FORMS

A. Plywood, wood, metal, glass, or a combination of these materials.

2.9 MISCELLANEOUS STEEL ITEMS

A. Galvanize or epoxy coat all miscellaneous steel items permanently cast into structural concrete elements. Refer to AASHTO M 111, and M284.

PART 3 EXECUTION

3.1 PREPARATION

A. Falsework

1. Construction:
   a. Use materials able to sustain the stresses required by the falsework design.
   b. Use suitable jacks or wedges to set the forms to the grade or camber required, and to prevent settling.
   c. Produce a finished structure of the specified camber, and built to the lines and grades indicated.
2. Footing Construction:
   a. Build falsework on a solid footing that is safe against undermining, protected from softening, and capable of supporting any imposed loads.
   b. Demonstrate that the soil bearing values do not exceed the supporting capacity of the soil. Conduct load tests or have soils investigation conducted by a professional engineer licensed in the State of Utah.
   c. Use piling or drilled shafts to support falsework that cannot be founded on a solid footing.
   d. Space, drive, and remove piles following approved falsework drawings.
3. Design and construct all falsework according to AASHTO LRFD Bridge Construction Specifications Section 3 (Temporary Works).

B. Forms

1. Use mortar-tight concrete forms, true to the dimensions, lines, and grades of the structure, and of sufficient strength to prevent deflection during the placement of concrete.
2. Discontinue using any form or forming system that produces a concrete surface with excessive undulations until modifications have been made. Undulations are excessive if they exceed either ⅛ inches or $^{1}/_{270}$ of the center-to-center distance between studs, joints, forms, fasteners, or wales.
3. Countersink all bolt and rivet holes when using metal forms for exposed

surfaces so that a plane, smooth surface of the desired contour is obtained.

4. Use lumber that is free of knotholes, loose knots, cracks, splits, warps, or other defects that affect the strength or appearance of the structure. Rough lumber may be used for forming surfaces if visible rough surfaces do not show on the final structure.
5. Form all exposed surfaces of each element of a concrete structure with the same forming material or with such materials that produce a concrete surface that is uniform in texture, color, and appearance.
6. Clean the inside surface of forms of all dirt, mortar, and foreign material before concrete placement.
7. Use form oil that permits the ready release of the forms and does not discolor the concrete.

8. Do not place concrete in the forms until:
   a. All work connected with form construction has been completed.
   b. All embedded materials have been placed.
   c. All dirt, chips, sawdust, water, and other foreign materials have been removed.
   d. Inspection and approval have been obtained.
9. Do not use stay-in-place deck forms unless otherwise specified.

C. Footings
   1. Excavation: Refer to Section 02317.
   2. The Engineer may direct written changes in dimensions or elevations necessary to secure a satisfactory foundation.
   3. Do not dewater by pumping during concrete placement, or for 24 hours thereafter, unless pumping is outside the enclosure. Do not use well points to dewater footing.

3.2 GIRDERS, SLABS, AND COLUMNS

A. Deck: Wet cure deck concrete at least seven days with continuously wet burlap or saturated thick cotton mats and until it has attained required design strength before placing parapet forms or leave all falsework in place and design it to carry all additional loads that are part of the parapet placement process.

B. Slab Span: Place concrete in one continuous operation.

C. Cast-In-Place T-Beams:

1. Place concrete in one or two continuous operations: The first to the top of the girder stems and the second to completion.
2. Obtain a bond between the stem and slab that is positive and mechanical, and secured by means of shear keys in the top of the girder stem.

D. Concrete in columns:

1. Allow footing concrete to set until it has attained 75 percent of its design strength based on field cylinder breaks before placing column forms when column is being placed on a footing.
2. Place concrete in one continuous operation.
3. Allow concrete to set at least two days before placing caps.
4. Do not place concrete in the superstructure until the columns have been stripped and approved.

E. Substructure Concrete: Do not place the superstructure load on the bents or abutments until they have been in place at least seven days or attained 75 percent of the design strength based on field cylinder breaks.

3.3 BOX CULVERTS

A. Allow base slab and footing to cure until they have both attained 75 percent of their design strengths based on field cylinder breaks before the remainder of the culvert is constructed.

B. Construct side walls and top slab monolithically unless the wall height exceeds 10 ft. Keep the construction joints vertical and at right angles to the axis of the culvert.

C. When side walls and top slab are not placed monolithically, construct shear keys in the top of the side walls for anchoring the top slab.

D. Construct wingwalls monolithically.

E. Do not backfill until all concrete has attained 100 percent of its required design strength based on field cylinder breaks.

3.4 CONCRETE SLOPE PROTECTION

A. Preparing subgrade:

1. Prepare the area to be paved by smoothing and shaping the berms and slopes and excavating for the cut-off walls.
2. Fill and compact all depressions and humps.
3. Furnish extra material to properly finish the slopes when required.
4. Compact all soft and yielding material resulting in a firm and substantial subgrade of uniform density.
5. Thoroughly sprinkle the area with water before placing the concrete.
6. Have the Engineer approve all surfaces before placing concrete.

B. Placing concrete:

1. Do not place concrete upon spongy, frozen, absorptive or unstable surfaces.
2. Provide concrete of a consistency that it can be placed on the slopes without deformation.
3. Complete all scoring as indicated on the plans.
4. Complete the entire slope protection in one placement if possible, or terminate the placement with a construction joint located in a scoring or at the junction of the slope and the abutment.
5. Finish concrete using a Floated Surface Finish according to this Section, article 3.11. Cure according to Section 03390.

C. Sealing joints and closures:

1. Furnish 1-inch thick, rigid plastic foam (styrofoam) for all expansion joints located between structural members and the slope protection.
2. Place the rigid plastic foam material against the surface of all structural members before placing the concrete slope protection.
3. Anchor the rigid plastic foam in place with a compatible adhesive or other approved methods.
4. Seal this area just before final inspection.
5. Remove curing compounds, oil, grease, dirt, and any other foreign

materials from concrete surfaces and grooves by sandblasting or other permitted methods.

6. Place the backer rod and sealant after the concrete has properly cured.
7. Apply the backer rod and sealant to clean and dry concrete surfaces.
8. Place sealant with hand or power-operated caulking guns after placing the backing materials. Refer to Section 03152.
   a. Limit the depth of sealant in the groove to ⅜ inch.
   b. Start the placement at one side and proceed to the other side on horizontal grooves and from top to bottom on vertical grooves.
   c. Use a concave pointing tool with soap solution to tool the sealant.
9. Do not place the sealant unless temperatures are at least 50 degrees F and rising.

D. Replacement:
   1. Prepare subgrade, place concrete and seal joints and closures per this Section, paragraphs A, B and C.
   2. Place concrete slope protection within seven days after removing damaged concrete slope protection. Refer to Section 03055.
   3. Connect reinforcement to existing concrete slope protection to remain in place as shown in the plans.
   4. Use a sealant that meets the requirements in Section 03152.

3.5 PLACE CONCRETE

A. Do not place concrete without approval.

B. Remove struts, stays, and braces that hold the forms in correct shape and alignment when no longer necessary.

C. Mix and transport concrete according to the limitations specified in Section 03055.

D. Do not deviate from the placement schedule without written approval.

E. The Engineer may postpone placement operations if the concrete cannot be protected during adverse weather.

F. Observe the following precautions when handling concrete:

1. Avoid segregation of the ingredients.
2. Arrange chutes, troughs, or pipes used as aids in placing concrete so the concrete does not separate.
3. Use metal or metal-lined chutes and troughs. Do not use aluminum.
4. Equip chutes with baffle boards or a reversed section at the end of the outlet when placing on steep slopes.
5. Extend open troughs and chutes down inside the forms or through holes left in the forms; terminate the ends in vertical downspouts.
6. Thoroughly flush all chutes, troughs, and pipes with water before and after each placement.
7. Do not allow the free-fall of concrete to exceed 10 ft for thin walls (maximum 10 inch thickness) or 5 ft for other types of construction without the use of a tremie or a flexible metal spout.
8. Use flexible metal spout sections composed of conical sections not more than 3 ft long, with the diameter of the outlet and the taper of the various sections such that the concrete does fill the outlet and retards concrete flow.

G. Observe the following precautions when placing concrete:

1. Deposit concrete as close as possible to its final position, without allowing it to flow laterally in the form.
2. Spread fresh concrete in horizontal layers with thickness not greater than what can be compacted with vibrators.
3. Do not use vibrators to flow concrete laterally.
4. Limit placement interruptions to 45 minutes.
5. Place and compact each layer before the preceding layer has taken initial set.
6. Do not place concrete in water flowing under head within the area of a footing.
7. Pass the screed over the area with a screed face device to measure the cover before concrete placement.
8. Relocate and tie reinforcing steel that projects above the specified level before placing the concrete.
9. Raise and support reinforcing steel that is more than ¼ inch below the

specified level before placing the concrete.

10. Firmly support screed rails for bridge deck slabs to prevent movement during concrete placement. When using a finishing machine, support the machine rails on the bridge beams. Do not place the machine rails on the forms unless the form supports have been strengthened and the Engineer gives written approval.

H. Observe the following precautions when compacting concrete:

1. Use high frequency internal vibrators to compact all concrete for structures except concrete placed under water.
2. Supply enough vibrators to compact the fresh concrete to the desired degree within 15 minutes after it is deposited in the forms.
3. Supply at least two vibrators for structures involving more than 25 yd^3 of concrete.
4. Do not attach vibrators to or against the forms or the reinforcing steel.
5. Do not allow vibrators to penetrate layers of concrete that have taken initial set.
6. Use spades or wedge-shaped tampers to secure a smooth and even texture of the exposed surface.

I. When using High Early Strength Concrete, verify that design strength has been obtained by field cylinder breaks.

3.6 PLACE CONCRETE UNDER WATER

A. Place and deposit concrete under water when specified on the plans.

B. Seal the forms or cofferdams watertight.

C. Do not pump water while placing concrete or disturb the concrete until it has set at least 24 hours, or attained at least 50 percent of its design strength.

D. Regulate placing to keep surfaces approximately horizontal at all times.

E. Place the concrete by beginning at one end of the form and progressing in a zig-zag movement from side to side across the length of the form.

F. Place the concrete using a tremie or concrete pumping equipment.

G. Observe the following steps when placing concrete with a tremie:

1. Use an 8-inch to 12-inch diameter steel tube tremie constructed with watertight connections, a hopper to receive concrete, and a device at the bottom to exclude water from entering the tube.
2. Use support that permits the discharge end to move over the entire top work surface and permits the tremie to be rapidly lowered to stop or retard flow when necessary.
3. Minimize the number of tremie location shifts for continuous placement.
4. Keep the tremie tube full to the bottom of the hopper during placement.
5. Slightly raise the tremie when a batch is dumped into the hopper, but do not raise it out of the concrete at the bottom until the batch discharges to the bottom of the hopper. If the concrete seal around the tube is lost, re-plug the end and refill the tube with concrete.

3.7 PUMP CONCRETE

A. Place concrete with a concrete pump in good operating condition. Replace pump that causes excessive or erratic loss of air entrainment.

1. Use a pump that produces a continuous stream of concrete without air pockets.
2. Do not add water to the concrete in the pump hopper.

B. Do not allow pump vibrations to damage freshly placed concrete.

C. Do not use concrete contaminated by the priming or cleaning of the pump.

3.8 LIMITATIONS

A. Light the work site so all operations are plainly visible if mixing, placing, or finishing occurs after daylight hours. Refer to Section 00555.

B. Keep all traffic off concrete bridges and culverts for 14 days after final concrete placement.

C. Cold Weather:

1. Cold weather limitations apply when the temperature is likely to fall below 40 degrees F within 14 days of placement.
2. Comply with the following regulations for placing concrete in cold weather:
    a. Submit a written plan for approval 14 calendar days before concrete placement.
    b. Do not use chemical additives in the concrete to prevent freezing.
    c. Provide all necessary cold weather protection for in-place concrete including cover, insulation, heat, etc.
    d. Do not place concrete in contact with frozen surfaces.
    e. Produce concrete with a temperature between 60 degrees F and 90 degrees F at the time of placing.
    f. Adequately vent combustion-type heaters that produce carbon monoxide.
    g. Maintain the concrete temperature above 50 degrees F and below 120 degrees F with no more than a 40 degree F temperature gradient at any one time for the first 14 days after placing.
    h. Protect the concrete from freezing until a compressive strength of at least 3,500 psi has been achieved.
    i. Maintain moist conditions for exposed concrete not in contact with forms; avoid loss of moisture from the concrete due to heat applied.
    j. Limit the drop in temperature next to the concrete surfaces when removing heat to 20 degrees F during any 12-hour period until the surface temperature of the concrete reaches that of the atmosphere.
    k. Determine the concrete temperature with a surface thermometer insulated from surrounding air.
    l. Remove and replace concrete damaged by frost action at no additional cost to the Department.
3. Heating Aggregate and Water:
    a. Provide and operate heating devices at no additional cost to the Department when heated aggregates are required.
    b. Aggregates must be free of ice.
    c. Heat aggregates uniformly, when required. Avoid overheating or

developing hot spots.

d. Use either steam or dry heat.

e. Combined water and aggregates in the mixer before the cement is added to avoid the possibility of a quick or flash set of the concrete when either the water or aggregates are heated to above 100 degrees F.

1) If this mixer-loading sequence is followed, water temperatures up to the boiling point can be used provided the aggregates are cold enough to reduce the final temperature of the aggregates and water mixture to less than 100 degrees F.

D. Hot Weather: Cool all form surfaces that will come in contact with the concrete to below 95 degrees F.

E. Hot Weather (Only Decks and Approach Slabs)

1. Begin placing concrete when the temperature is declining.
2. Begin batching operations when the air temperature in the shade is 85 degrees F or less.
3. Discontinue placing when the temperature reaches 80 degrees F in the shade and is increasing.

F. Surface Evaporation:

1. Surface evaporation limitations apply and may occur at any time of the year, when any combination of air temperature, relative humidity, and wind velocity, that have the potential to impair the quality of fresh or hardened concrete or otherwise result in abnormal properties. Submit a written plan for approval 14 calendar days before concrete placement that shows proper attention will be given to ingredients, production methods, handling, placing, protection, and curing to prevent excessive concrete temperatures and water evaporation that could impair strength or serviceability of the concrete. Refer to ACI 305.
2. The surface evaporation plan may include any of the following actions:

a. Construct windbreaks or enclosures to effectively reduce the wind velocity throughout the area of placement.

b. Use fog sprayers upwind of the placement operations to effectively

increase the relative humidity.

c. Reduce the temperature of the concrete by shading the material storage area or production equipment, cool aggregate by sprinkling, cool aggregate or water by refrigeration or by replacing a portion or all of the mix water with flaked or crushed ice to the extent that the ice will completely melt during mixing of the concrete.

d. Adjustment of the placement schedule.

e. Use an approved water-based mono-molecular polymer liquid evaporative reducer at application rates recommended by the manufacturer. Do not use as a finishing aid.

3.9 EXPANSION JOINTS AND BEARINGS

A. Refer to Section 05832 for expansion joint information.

B. Refer to Section 05822 for bearing information.

C. Adjust bearing positions and joint widths as shown on plans.

3.10 CONSTRUCTION JOINTS

A. Make construction joints where shown on plans or in the placing schedule.

B. Obtain Engineer's written approval when additional construction joints are desired and meet the following requirements:

1. Place and construct without impairing strength and appearance.
2. Place in planes perpendicular to the principal lines of stress and at points of minimum shear.
3. Make monolithic structures by extending the reinforcing across the joint.
4. Avoid construction joints through paneled wing walls or large surfaces which are to be treated architecturally.
5. Make a straight line joint across the face of the pour for the full width of the bridge deck.
6. Leave a rough surface to increase the bond with the concrete placed later.
7. Form tapered sections with an insert so that the succeeding layer of

concrete ends in a section at least 6 inches thick.

8. Place a bulkhead from the surface to the top mat of steel to ensure a straight vertical face. Shape the concrete below the top steel to a near vertical face in line with the bulkhead.
9. When a bulkhead cannot be placed, establish a straight vertical face by saw cutting to a minimum depth of 1 inch. Shape the concrete below the saw cut to a near vertical face.

C. Before resuming concrete placement, meet the following:
   1. Re-tighten forms.
   2. Roughen the surface of hardened concrete without leaving loosened particles or damaged concrete.
   3. Clean off concrete surface of foreign matter and laitance by sandblasting.
   4. Saturate concrete surface with water.
   5. Apply epoxy adhesive as specified to face of construction joints.

3.11 CONCRETE SURFACE FINISHING CLASSIFICATIONS

A. Ordinary Surface Finish: A true and uniform finished surface.

B. Rubbed Finish: A surface smooth in texture and uniform in appearance, free of all form marks or irregularities.

C. Wire Brush or Scrubbed Finish:
   1. A finished surface with the cement surface film completely removed and the aggregate particles exposed leaving an even-pebbled texture.
   2. An appearance ranging from fine granite to coarse conglomerate depends on the size and grading of the aggregate used.

D. Floated Surface Finish:
   1. Flat work: strike off and use a floated surface finish.
   2. Bridge decks and approach slabs: machine finish only.

3.12 CONCRETE SURFACE FINISHING

A. Give all formed concrete surfaces at least an Ordinary Surface Finish except as specified otherwise.

B. Use other types of finishes as required in addition to the Ordinary Surface Finish.

C. Provide a Rubbed Finish for all surfaces that cannot meet Ordinary Surface Finish requirements due to irregularities, honeycombing, excessive surface voids, discoloration, and other defects.

3.13 CONCRETE SURFACE FINISHING PROCEDURES

A. Ordinary Surface Finish:

1. After removing forms, remove all fins and projections.
   a. Clean, point, and true all honeycomb spots, broken corners or edges, cavities made by form ties, and other holes and defects.
   b. Keep all areas to receive mortar saturated with water for at least 30 minutes before mortar placement.
2. For pointing, use a mortar of cement and fine aggregate, not more than one hour old, mixed in the proportions used in the grade of concrete being finished.
3. Cure the mortar patches and rub to blend with surrounding concrete.
4. Tool and free all joints of mortar and concrete. Leave the full length of the joint filler exposed with clean and true edges.

B. Rubbed Finish:

1. Wet the surface of concrete while still green, paint with grout, and rub with a wooden float until the surface is covered with a lather of cement and water.
   a. A thin grout of one part cement, one part fine sand may be used in the rubbing.
   b. Let this lather set for at least five days, then rub lightly with a fine carborundum stone until smooth.
2. For hardened concrete, use a mechanically operated carborundum stone to finish the surface at least four days after placing.

a. Finish in the same manner as above; however, let the lather set for at least 15 days before lightly rubbing with a fine carborundum stone until smooth.

3. Commercial grade rubbing mortar may be used if approved by Engineer.

C. Wire Brush or Scrubbed Finish:

1. After the forms are removed and the concrete is green, scrub the surface with stiff wire or fiber brushes using a solution of muriatic acid – one part acid, four parts water.
2. Once the scrubbing produces the desired texture, wash the entire surface.
3. Use water mixed with 5 percent by volume ammonium hydroxide to remove all traces of the acid.

D. Floated Surface Finish on flat work other than bridge decks and approach slabs:

1. Striking Off:
   a. After compaction, carefully rod and strike off the surface with a strike board following the cross sections and grades shown on the plans.
   b. Allow for camber as required.
   c. Operate the strike board longitudinally or transversely and move it forward with a combined longitudinal and transverse motion, ensuring that neither end is raised from the side forms during the process.
   d. Keep a slight excess of concrete in front of the cutting edge at all times.
2. Floating:
   a. Use longitudinal, or transverse floating, or both to create a uniform surface.
   b. Longitudinal floating is required except in places where it is not feasible.
3. Longitudinal Floating:
   a. Work the longitudinal float, operated from foot bridges, with a sawing motion while holding it parallel to the road centerline.
   b. Pass gradually from one side of the pavement to the other. Move the float forward one-half of its length and repeat operation.
   c. Substitute machine floating, if equivalent results are produced.

4. Transverse Floating:
    a. Operate the transverse float across the concrete surface by starting at the edge and slowly moving to the center and back again to the edge.
    b. Move the float forward one-half of its length and repeat the operation.
    c. Preserve the crown and cross section of the concrete surface.
5. Straightedging:
    a. Test the concrete surface for trueness with a straightedge after the longitudinal floating has been completed and the excess water has been removed, but while the concrete is still plastic.
    b. Furnish and use an accurate 10 ft straightedge held parallel to the road centerline in contact with the surface.
    c. Check the entire area, immediately filling depressions with freshly mixed concrete, then strike off, consolidate, and refinish.
    d. Cut down and refinish high areas.
    e. Continue the straightedge testing and re-floating until the concrete surface is at the required grade and contour.

E. Floated Surface Finish for bridge decks and approach slabs:
1. Machine-finish exposed surfaces unless otherwise permitted.
2. Finish concrete by striking off and floating the surface.
3. Allow the Engineer enough time to inspect finishing machines during daylight hours before concrete placement.
4. Stop finishing operations hampered by darkness unless lighting facilities are provided.
5. Extend finishing machine rails beyond both ends of the scheduled placement, and allow sufficient distance to permit the float to fully clear the concrete.
6. Use adjustable rails set to elevations established by the Engineer, installed to prevent springing or deflection under the weight of the finishing equipment, and placed to operate without interruption.
7. Place screed machine parallel to the abutments and bents within 10 degrees.
8. Support screed rails to prevent movement during placing of the concrete.
9. Either support finishing machine rails on the bridge beams or on form

supports stiffened to prevent deflection.

a. Obtain written approval before using form supports.

b. This may require load tests.

10. Attach a measuring device to the screed face and pass it over the area.
11. Before placing concrete, relocate and tie reinforcing steel that projects above the specified level, and raise and support steel that is more than ¼ inch below the specified level.
12. Place concrete in a uniform heading approximately parallel to the screed machine.
13. Limit the rate of placing to allow enough time to finish the surface before initial set.
14. Continuously place concrete the full length of the structure or superstructure unit unless otherwise shown or approved.
15. Provide sufficient material, equipment, and manpower to place deck concrete at a minimum rate of 25 yd^3/hour.
16. Strike off the surface to the required elevations with the finishing machine immediately after placing and consolidating the concrete.
17. Do not add water to the concrete in front of or behind the screed.
18. Have the strike-off method and equipment approved. Maintain satisfactory performance. Use equipment capable of finishing concrete within the surface tolerances specified. Maintain satisfactory consolidation and surface tolerance to prevent shutdown and rejection of the equipment.
19. Furnish a 10 ft straightedge to check the surface tolerance, placed both longitudinally and transversely, immediately behind the screed machine and hand-finished areas.
20. Correct irregularities greater than ⅛ inch from the straightedge, before additional placement, and immediately fill depressions with concrete, and refinish.
21. Cut down and refinish high areas.
22. Continue straightedge testing and corrective measures until the entire surface is free of observable departures from the straightedge.

F. Final texturing for bridge decks and approach slabs: (a textured hardened finish):

1. Do not texture finish concrete deck surfaces after floating that will be covered by a water-proofing membrane system.

2. Use a texture process that produces regular ⅛ inch wide transverse grooves spaced randomly from ½ inch to ¾ inch on centers and ⅛ inch deep.
3. Keep the finished surface free from porous spots and surface irregularities.
4. Furnish a work bridge that follows the finishing machine to facilitate texturing and application of the membrane-curing compound.
5. Check the surface smoothness for acceptance after the concrete has hardened.
6. Remove irregularities by grinding if the surface deviates more than ⅛ inch from a 10 ft straightedge. Refer to Section 02752.

3.14 CURE

A. Refer to Section 03390.

3.15 FORM REMOVAL

A. Obtain approval before removing forms.

B. Remove all forms from the concrete surfaces.

C. Do not use any method of form removal likely to cause overstressing of the concrete.

D. Remove supports to permit the concrete to uniformly and gradually take the stresses due to its own weight.

E. Do not remove forms used in ornamental work, railings, parapets, and exposed vertical surfaces for at least six hours after placement.

F. To determine the condition of columns, always remove forms before removing shoring from beneath beams and girders.

G. Removing falsework:
   1. Do not remove deck falsework until the backfill at the abutments has been placed up to the bottom of the approach slab.

2. Do not remove falsework supporting the deck of rigid frame structures until the fill has been placed in back of the vertical legs.
3. Keep falsework and forms in place under slabs, beams, and girders for 14 days after the day of last concrete placement. Slab forms with a clear space of less than 10 ft may be removed after seven days.
4. In cold weather, keep forms and falsework in place as approved in the written plan for cold weather concrete.

H. Patch formed surfaces within 24 hours after form removal:
1. Cut back and remove all projecting wire or metal devices used for holding the forms in place and that pass through the body of the concrete at least 1 inch beneath the surface of the concrete.
2. Remove lips of mortar and all irregularities caused by form joints.
3. Fill all small holes, depressions, and voids with cement mortar mixed in the same proportions as that used in the body of the work.
4. To patch larger holes or honeycombs, obtain a solid uniform surface by chipping away coarse or broken material.
   a. Cut away feathered edges to form faces perpendicular to the surface.
   b. Cover with epoxy-adhesive coating as specified. AASHTO M 235, Type II
   c. Fill the cavity with stiff mortar composed of one part portland cement to two parts sand thoroughly tamped into place.
   d. Pre-shrink the mortar by mixing it approximately 20 minutes. Vary the time according to manufacturer's recommendations, temperature, humidity, and other local conditions.
   e. Float the surface of this mortar with a wooden float before initial set.
   f. Keep the patch wet for five days.
   g. After curing, rub patches on exposed surfaces to blend them with surrounding concrete.
   h. Add coarse aggregate to the patching material when patching large or deep areas.
   i. Make a dense, well-bonded, and properly cured patch.

I. Areas with extensive honeycombing will be rejected. After receiving written

notice of rejection, remove and rebuild the structure in part or wholly, as specified, at no additional cost to the Department.

J. Apply the following requirements after fully removing all the closure joint forms if inserts are placed along the bottom edges of the precast concrete deck panels to form the closure pour joints:

1. Cut off cast-in-place anchors at least 1inch below the face of slab and repair per this Section, article 2.2.
2. Fill all voids with dry-pack mortar flush with the bottom of slab.
3. Fill voids created by the removal of re-usable concrete anchors with dry-pack mortar flush with the bottom of slab.
4. Dry-pack mortar will be composed of one part portland cement to two parts sand.

3.16 MISCELLANEOUS CONSTRUCTION

A. Drainage and weep holes:

1. Construct drainage and weep holes at locations indicated on the plans or as directed.
2. Place ports or vents for equalizing hydrostatic pressure below low water.
3. Use non-corrosive materials for weep hole forms.
4. Remove wooden forms after the concrete is placed.
5. Paint exposed surfaces of metal drains as indicated on the plans.

B. Anchor Bolts: Securely and accurately set all necessary anchor bolts in piers, abutments, or pedestals as the concrete is being placed.

C. Bearing plate areas:

1. Finish bridge seat bearing areas high and rub or grind to grade level within an allowable tolerance of $\pm^{1}/_{16}$ inch within a tolerance of ± ⅛ inch of the elevation shown on the plans.
2. Do not grout under bearing plates.

3.17 CLEAN

A. Clean up by removing all falsework and falsework piling down to 2 ft below the

finished ground line, rubbish, and temporary building materials before final inspection.

END OF SECTION

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