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Concrete Diffusion Coefficients and Existing Chloride Exposure in North Carolina

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16. Abstract

The present project focused on the diffusion coefficients of ten concrete mixes used in North Carolina bridge structures, and on the chloride loading rates present in the different climatic and environmental regions of the state with respect to the mitigation policies and structural condition of bridges in North Carolina. In order to determine the diffusion coefficients of the selected mixes, ponding tests were performed for 12 months. These results were then correlated with rapid chloride permeability tests, and later with bulk diffusion and rapid migration test results. Furthermore, as a possible alternative to the potentiometric titration, a rapid chloride test kit was also evaluated to determine the chloride content of concrete powder samples.

A survey was sent out to highway and bridge maintenance personnel in the six geographic regions considered (Coastal, Manteo, Triad, Triangle, Piedmont and Mountain regions), requesting information with regards to deicing material type and frequency/quantity of application on bridges/roads. To supplement this information, 28 bridges were sampled, focusing on bridge decks and pier caps/abutments for inland bridges; and footings, columns and decks for coastal bridges. An attempt was also made to develop a corrosion model using chloride surface concentrations, estimated diffusion coefficients, and age of structure. This model was also calibrated using historic data developed for a few coastal bridges.

17. Key Words
Chloride diffusion, RCPT, rapid migration test, rapid chloride test, bulk diffusion test

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The authors would like to thank the division personnel from the six geographic areas considered for their technical, equipment and material assistance throughout the project. Their assistance included bridge identification, site preparation, traffic control, access to boats, and other materials and supplies for the 28 bridges sampled. Many of the materials for the 10 mix designs have been donated to this project by the distributors and manufacturers. Furthermore, the environmental chamber was provided at cost by W.A. Brown, and technical on-site assistance was provided by Vincent Washington of Master Builders. Their contributions are greatly appreciated.

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EXECUTIVE SUMMARY

The present project consisted of two lines of research: one to determine the diffusion coefficients of ten concrete mixes used in North Carolina bridge structures, and correlate these numbers with RCPT results; and to determine the chloride loading rates present in the different climatic and environmental regions of the state, and to analyze the mitigation policies and structural condition of bridges in North Carolina.

The preset project established, using the ponding test, chloride diffusion coefficients for 10 mix designs widely used in the state of North Carolina. Parallel to this effort, rapid chloride permeability tests (RCPT) have also been performed on all mix designs. In order to measure the chloride content of the powder samples retrieved during sampling of the ponding slabs, potentiometric titration has been used. The original proposal has been amended to include an investigation into the rapid chloride test (RCT), which was found to be a reliable and more practical alternative, and could also be used in the field.

Through the results of this project it has been shown that there are some reasonable correlations between the salt ponding test and the RCPT results for the mix designs without supplementary cementitious materials. However, when supplementary cementitious materials and corrosion inhibitor admixtures are added to the mix designs, the correlation becomes much weaker. In order to address this, some of the mix designs were tested with two relatively new permeability tests as well: the bulk diffusion and rapid migration tests. These two tests provide alternatives to the salt ponding test and the RCPT, respectively. The results obtained from the tests provided good correlations with each other, as well as with the ponding and RCPT tests.

It has been also shown that the principles of diffusion can be used to reasonably predict the concentration of chloride in reinforced concrete bridge elements. A model that uses Fick’s Second Law of diffusion can estimate the concentration of chloride at the depth of the reinforcing steel after a given amount of time if accurate estimates of the material properties (diffusion coefficient) and environmental conditions (chloride loading) are available. This information can be used with estimates of corrosion threshold to make service life predictions based on corrosion induced deterioration. It

was shown that the modeling procedure used to design the Virginia Dare Bridge is currently valid. Using historic chloride content information, the model was tested for its ability to accurately predict chloride concentration based on information gathered for this research project. These reverse predictions were accurate with a range of error between 20% and 46%.

Field research was conducted in inland areas to determine the necessity of a corrosion design policy for bridges that are subject to road salting as their primary exposure to chlorides. This research phase consisted of results from a survey of road maintenance engineers as well as a field sampling program which tested bridge elements for their chloride content. Results from these studies indicate that there is significant exposure to chlorides in regions across the state. Unlike bridges on the coast for which the source of chloride is present regardless of the bridge usage, inland bridges are exposed to chloride in a way that is proportional to their use.

The results of the survey verified that the Mountain and Triad roads are exposed to considerably more salt on an annual basis than roads in other parts of the state; and therefore, bridges in these areas should be specially treated in the chloride mitigation policy. It is also important to note that roads not included in the Bare Pavement System are salted in the Mountain region at comparatively high rates. Some Secondary Roads in the mountains are being salted at rates equivalent to Bare Pavement routes in other regions! This indicates that special attention should be paid to bridges on less traveled roads as well.

Surprisingly, the Triad region showed the highest surface chloride concentrations when non-coastal bridges were considered, even though the survey suggested a higher deicing salt application rate in the Mountain region. Furthermore, the results of inland bridge samples showed that, with only few exceptions, pier caps and abutments have comparable chloride contents (at most of the depths investigated) as bridge decks. This suggests that the chloride mitigation policy should include provisions for these structural elements as well, and provide corrosion protection through special concrete mix designs and rebar types or coatings.

Further studies are recommended in order to provide a larger sample size for the mix designs considered, and the concrete bridge components tested.

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1. INTRODUCTION

1.1 Introduction

The availability, flexibility and economy of reinforced concrete have made it a ubiquitous material in heavy construction of all sorts. A large proportion of the bridges in North Carolina’s highway network are composed of reinforced concrete structural members. The service life and safety of these concrete bridges can be seriously reduced if the reinforcing steel is compromised by corrosion. The chemical processes that precede corrosion are accelerated by the presence of chloride ions that are introduced to the concrete matrix via sea water in marine environments or by road salting in climates that require ice removal.

A challenge for bridge designers and maintenance personnel is to mitigate the effect of the chloride ingress by increased protection of the steel, or by preventative measures and condition inspections of existing bridges. Engineers have spent decades researching and developing concrete mix designs and admixtures to resist the diffusion of chloride ions into concrete. These designs included such products as fly ash, blast furnace slag, and silica fume, all by-products of energy production, to increase the density of concrete. These admixtures show the most promise in reducing the permeability of the cured concrete.

It is known that the permeability of concrete is based on multiple material properties including aggregate type and gradation, cement content, and use of secondary cementitious materials. Chloride ingress can be modeled mathematically with Fick’s second law of diffusion; however, this method requires experimental determination of a diffusion coefficient.

The coefficient is most accurately determined by way of a long term test during which concrete slabs are subjected to continuous ponding with a salt solution having a known depth and salinity. After the ponding period, powder samples are retrieved from specified depths in the concrete slabs and the chloride concentration is measured. The test procedure just described is given by the American Association of State Highway and Transportation Officials (AASHTO) as test procedure T259, “Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration” (AASHTO, 2002).

While the T259 method provides an accurate diffusion coefficient for predicting chloride permeability, its application is time consuming, especially for very low permeability concretes. The duration of a single test can exceed 1 year. A more rapid method is put forth by the American Society for Testing and Materials (ASTM), the ASTM C1202, “Electrical Indication of Concrete’s Ability to Resist Chloride Ion Penetration” (ASTM, 1997).

Often referred to as the Rapid Chloride Permeability Test (RCPT), the method can provide permeability information in six hours. The rapid test is a direct measurement of concrete’s resistance to electrical current and only an indirect measurement of chloride permeability. Its results are also confounded by some material properties as well as by the age of the concrete. The results can be rendered more useful with a means of correlation to ponding test data.

A second method of determining the diffusion coefficient of concretes is applicable to existing structures in the field. In this method, chloride content profiles are analyzed mathematically to determine the diffusion coefficient that would create the profile given a specific loading arrangement and time.

1.2 Research Objectives

The present project consisted of two lines of research: one to determine the diffusion coefficients of ten concrete mixes used in North Carolina bridge structures, and correlate these numbers with RCPT results; and the other, to determine the chloride loading rates present in the different climatic and environmental regions of the state, and to analyze the mitigation policies and structural condition of bridges in North Carolina.

After acceptance of the project proposal, a final element was added to these tasks. A simplified chloride testing method is available in the form of a portable device produced by Germann Instruments. The device was evaluated for its efficacy and accuracy by using it in parallel with the accepted laboratory titration method given in AASHTO T260 (AASHTO 2002). In this paper the performance of the device is reported and recommendations on possible use in the future are made.

The initial proposal to the NCDOT stated that both the ponding test and RCPT would be performed on the selected mix designs. To further supplement the project, and to improve correlation results, two additional tests were performed on 4 of the ten mix designs. These tests were the ASTM C1556-03, Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion (ASTM, 2003), and NT Build 492, Chloride Migration Coefficient from Non-Steady-State Migration Experiments (Nordtest, 1999). Correlations were made between these two tests and the ponding and RCPT results.

The second phase of this project focused on field data which was gathered from bridge sampling across the state of North Carolina, and a survey of NCDOT personnel involved with road salting.

2. LITERATURE REVIEW AND BACKGROUND INFORMATION

In order to better understand chloride the diffusion in concrete, the implications of design requirements and maintenance policies, the corrosion mechanisms and related issues will be reviewed first, followed by testing methods and information available on correlation techniques. Finally, available information is reviewed on chloride profiles in concrete bridges and on corrosion threshold.

2.1 Chloride Diffusion in Concrete

Chloride ions are transported through the concrete matrix via several pathways. These include diffusion, capillary absorption, and hydrostatic pressure (Hooton et al., 2001). Absorption takes place during wetting and drying cycles. During these cycles, chlorides are absorbed by the suction of water containing the chlorides into the concrete pores. Chloride ions are also introduced by hydrostatic pressure, or by standing water, which causes the permeation of chloride ions through the matrix. Diffusion is the mechanism that is capable of bringing chlorides to the level of the reinforcing steel, thereby accelerating the corrosion of the rebar.

Chloride diffusion into concrete is described by Fick's second law, Equation 2-1:

$$\frac{\partial C(x)}{\partial t} = D_c \frac{\partial^2 C}{\partial x^2} \qquad (2\text{-}1)$$

where:

C=concentration of a chemical species [lbs/yd^3 or kg/m^3]

x=a linear distance [in. or meters]

t=time [seconds]

D_c=diffusion coefficient [in^2/year or m^2/second]

The concentration of chloride ions, or any other chemical species as a function of depth and time, is described by the solution to this relationship, Equation 2-2:

$$C(x,t) = C_0\left\{1 - erf\left(\frac{x}{2\sqrt{D_c * t}}\right)\right\} \tag{2-2}$$

where:

C_0 =the surface concentration of a chemical species

erf =the error function

This solution is based on the following conditions: 1) The surface concentration, C_0, is constant with time; that is, $C_{(x=0,\, t>0)} = C_0$. 2) The initial concentration is zero, $C_{(x=0,\, t=0)} = 0$. 3) The concentration at an infinite distance away from the surface is zero, $C_{(x=8\,,\, t>0)} = 0$. If these conditions are satisfied, then Equation 2.2 can be used.

The diffusion of chloride in concrete is more complex than the process defined by Fick's Second Law. In addition to the binding of chemicals, there are interactions between ions present in the concrete, and anions that cause a lagging action to occur (Zhang and Gjørv, 1996). This complexity is known, but is typically ignored to simplify the diffusion coefficient calculation.

2.2 Rapid Chloride Permeability Test

The rapid chloride permeability test (RCPT) was developed in 1981 by David Whiting for the Federal Highway Administration, FHWA (Whiting, 1981). It was developed to provide an alternative to the salt ponding test, which is a long-term test. In his original report, Whiting outlined two different procedures: one for field testing and one for laboratory testing. The field testing method was not as practical as the laboratory method because one lane of traffic had to be closed for five days and the correction factors for the depth of cover and ambient temperature were only known for conventional concretes.

The basic principle behind the RCPT is the applied voltage technique. This technique is based on the principle that a charged ion, such as chloride ions, will migrate in an electric field in the direction of the pole of the opposite charge. The test requires that 60 V_{DC} be applied to a concrete cylinder 4" in diameter and 2" thick for 6 hours. The amount of coulombs passed through the cylinder is then used to rate the concrete according to the standard's rating system, seen in Table 2-1.

Table 2-1. Interpretation of RCPT Results

Charge Passed	Chloride Ion Penetrability
>4,000	High
2,000-4,000	Moderate
1,000-2,000	Low
100-1,000	Very Low
<100	Negligible

Whiting (1981) showed that the RCPT results correlated very well with ponding test results from the same concrete mixtures. This correlation was based on the total integral chloride content of the ponded specimens. The total integral chloride is calculated by finding the area beneath the chloride content versus depth curve. An example of this procedure is shown in Figure 2-1 (Hooton and McGrath, 1999). Although this procedure provided good correlation for the concrete mixes tested by Whiting, it does not provide high correlations with concretes with admixtures or supplementary cementitious materials. These concretes are typically known as high performance concretes (HPC).

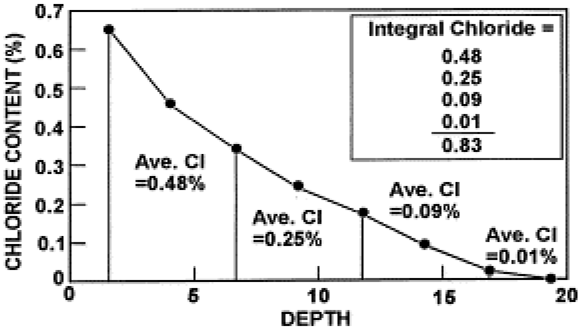


Figure 2-1. Total integral chloride content (from Hooton et al., 2001) (25.4 mm=1 in.)

There are questions as to whether or not the RCPT can accurately predict the permeability of concrete mixes containing mineral admixtures such as silica fume and fly ash (Suryavanshi et al., 2000). Silica fume and fly ash are both manufacturing by-products. Silica fume comes from the manufacture of silicon and ferrosilicon alloys, and fly ash is produced from the combustion of coal for energy production. These materials are used to replace or to supplement the cement in a concrete mixture, providing a denser concrete matrix, thereby reducing the permeability of the concrete. The reduction has been proven time and time again through long-term ponding tests, but the results from the RCPT are not conclusive. This is due to the variables introduced with the addition of these mineral admixtures, including the pore fluid conductivity and the micro-structural characteristics of the concrete (Suryavanshi et al., 2000).

The addition of these admixtures causes an increase in the amount of charge passed during the RCPT test, indicating a higher permeability than the ponding test. This is because the RCPT measures the movement of all the ions in the pore solution, not just the chloride (Joshi and Chan, 2002). Therefore, it is not clear how accurately the charge passed data from the RCPT can be used to determine the concrete's permeability for mixes containing mineral admixtures (Suryavanshi et al., 2000).

Mineral admixtures are not the only concrete additives that disrupt the RCPT. Nitrite-based corrosion inhibitors also cause unduly high RCPT values. These inhibitors are used to fend off corrosion of rebar through chemical attack. Based on tests completed by Joshi and Chan (2002), two different corrosion-inhibiting admixtures were tested with the RCPT, as well as a control concrete. The concrete with the admixtures had 2470 and 3209 coulombs passed, compared to 1211 coulombs for the control concrete. A separate report by Loulizi et al. (2000) also found that concrete mixes with corrosion-inhibiting admixtures had higher RCPT values than mixes without those admixtures.

2.3 Salt Ponding Test

The salt ponding test is the most widely accepted test method for determining the chloride permeability of concrete. There are two versions of this test: AASHTO T259 and ASTM C1543 (AASHTO & ASTM, 2002). The AASHTO test consists of 42 days of preparation and 90 days of ponding. The ASTM method lasts for a subjective length of time determined by the concrete type. Both tests require a 3% salt solution to be ponded on concrete slabs measuring 12" square by 3" thick. This solution is changed every two months for the ASTM method, while it is not changed for AASHTO. The chloride concentration is determined for 0.5-inch slices of the slab.

There are a number of critics of the salt ponding test that point out several shortcomings. One of these shortcomings is determining what the results from the chloride concentration measurement actually mean (Hooton et al., 2001). This complication is due to the crudeness of the testing conditions. It is hard to develop a true chloride profile from these results. Instead, this method gives an average chloride concentration over a 0.5” section.

Hooton suggests a scenario in which this could cause a problem. Assume there are two different concrete mixes, A and B, which were exposed to the salt solution as outlined in the standard. When these two slabs are tested, the average chloride concentration of each is the same for the first 0.5”; however, one of the mixes has a constant chloride profile for the half inch, mix B, while the other has a high concentration at the surface and then tapers off at the end of the half inch, mix A. Obviously the former concrete would reach a critical chloride concentration at the rebar level prior to the later, but this cannot be seen with a 0.5” sample. Figure 2-2 illustrates this problem.

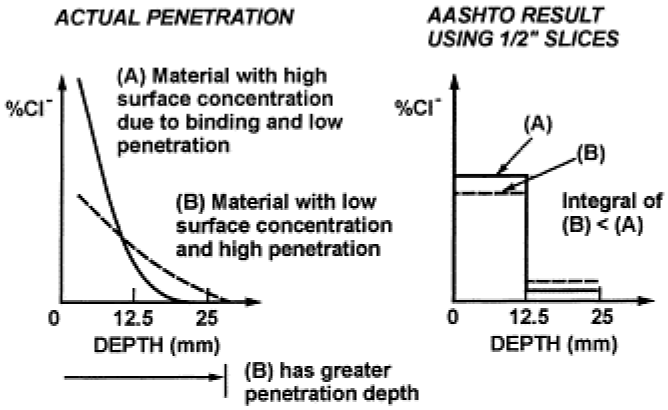


Figure 2-2. Illustrating the problem with the AASHTO sampling technique (from Hooton et al., 2001) (25.4 mm = 1 in.)

Another issue with the ponding test is that it allows for other methods of chloride ingress. This test method was developed to estimate the permeability of a concrete mix based on its resistance to the diffusion of chloride ions into the concrete matrix. This test allows for other transport mechanisms including sorption and wicking. The test procedure calls for 28 days of drying time before the salt solution is added. When the solution is finally added, there will be suction of the chloride solution into the pores of the mix due to the wetting effect (Hooton et al., 2001). Also, the bottom face and sides of the slab are exposed to 50% relative humidity, which causes more water to be drawn into the concrete, bringing chloride ions with it. This transport mechanism is known as wicking.

There have been several suggestions to make the ponding test better. These include monitoring and controlling the salt solution concentration, eliminating the partially saturated condition of the samples, and to use a profile grinding sampling technique (Hooton et al., 2001). The ASTM standard addresses the issue of controlling the salt solution by requiring the solution to be changed every two months if a period of time longer than 90 days is being tested (ASTM, 2002). The AASHTO method only requires that the level of the solution be kept constant throughout the test period. It is easy to see the difficulties created by this procedure. This will cause an increase in the salt concentration in the solution, which could lead to erroneous results because the test is based on a 3% ponding solution (AASHTO, 2002).

2.4 Rapid Migration Test

The Rapid Migration Test (RMT) was developed by Tang and Nilsson at Chalmers Technical University in Sweden. The test that they developed was adopted as a Nordtest Method, NT Build 492 (Nordtest, 1999). This test method is similar to the RCPT in that a 2-inch thick cylinder with a 4-inch diameter is subjected to an applied voltage for a period of time. The difference in this test is the length of time, typically 24 hours, and the voltage used, ranging from 10-60 V_{DC}. This test method has been suggested to be a better option than the RCPT test for a wider variety of concrete mixes (Hooton et al., 2001).

The RMT test was developed to address some of the problems of the RCPT test (Nordtest 1999). One of the major benefits of this test is that it allows for the calculation of a nonsteady-state diffusion coefficient. This diffusion coefficient is a function of the applied voltage, temperature of the solution, thickness of the specimen and the depth of chloride penetration. Hooton et al. conducted a project in which they tested different concrete mixes using the RCPT test, the RMT test and the bulk diffusion test (Hooton et al., 2001). In this test they found that correlation between the RMT results and the bulk diffusion results were equal to or better than the RCPT correlation. Also, the use of corrosion inhibiting admixtures did not affect the RMT results like it does with the RCPT results. This suggests that the RMT is capable of testing a wider range of concretes than the RCPT.

2.5 Bulk Diffusion Test

The bulk diffusion test was developed to address some of the short-comings of the AASHTO salt ponding test. The original standard was Nordtest NT Build 443 (2003), and ASTM later adopted the standard as ASTM C1556 in 2003 (ASTM, 2003). The test is performed on a cylinder that is epoxied on all sides except the top surface. This specimen is then saturated with limewater until the total weight of the specimen does not change by more than 0.1% over the course of 24 hours (ASTM, 2003). This saturation allows the sample to undergo a purer diffusion process than the ponding test because there is no initial sorption due to the drying effect (Hooton et al., 2001).

To determine the chloride content of the sample, the specimen is mounted into a lathe or a mill and sampled using a diamond-tipped core drill bit. The sampling depths begin at 0.039” (1 mm) and continue until an accurate chloride profile is obtained (ASTM, 2003). A minimum of 8 depths must be sampled and tested. The chloride content of the concrete is then determined using AASHTO T260 (Hooton et al., 2001).

The bulk diffusion test does address some of the issues with the salt ponding test such as limiting the effect of sorption and wicking on the sample, but it is still a long-term test. The minimum amount of time the specimen is to be exposed to the salt solution is 35 days and for high performance concretes, the exposure time should be increased to at least 90 days (Hooton et al., 2001). The advantage to the bulk diffusion test is that after 90 days, a good chloride profile can be developed from even the densest concretes, whereas the ponding test could require up to a year, or more.

2.6 Correlation Between Rapid and Long-term Tests

Long-term tests such as the salt ponding test and bulk diffusion are the only known accurate ways to determine the true chloride permeability of a concrete mix. The problem with these tests is the length of time required to get a "definitive" answer. The ponding test can last as long as a year for high performance concretes, as well as the bulk diffusion test. This is the reason quick tests like the RMT and RCPT were developed. In order to use the results from these rapid tests, correlations had to be made between them and the accepted long-term tests. There have been many papers published on this topic, listing many ways to perform this correlation. The correlation techniques that were considered for this project will be reviewed here.

Since the RCPT was introduced in 1981, there have been many attempts to correlate the charge passed data to the data from the salt ponding test. Whiting (1981) obtained good correlation between the charge passed from the RCPT and the total integral chloride content of the concrete subjected to the ponding test. The problem with Whiting's findings was that the correlations were not done on HPCs. The correlation between the total integral chloride and the charge passed for HPCs has been found to be not as good as Whiting's original research suggests (Myers et al., 1997). Myers et al. found that the correlation between the chloride content within the top 0.5" of the ponded specimen and the charge passed was better than the comparison between the total integral chloride content and charge passed, with an r^2 of 0.71 to an r^2 of 0.63, respectively. The r^2 value here simply indicates how well the regression line estimates real data points. An r^2 value of 1.0 (100%) indicates a perfect fit.

Another method of correlation is to calculate a diffusion coefficient from the RCPT results to be compared to the coefficient obtained from the ponding test (Jonsson, 2003). The diffusion coefficient from the RCPT is calculated based on Equation 2-3, or Equation 2-4 if the concrete mix has a calcium nitrite-based corrosion inhibitor. The results from this paper indicated a good correlation between the RCPT and salt ponding diffusion coefficient. Good correlation has also been found using the depth to 0.1% chloride content of the ponded specimens to the charge passed from the RCPT (Hooton, et al., 2001). Hooton et al. also suggests shortening the RCPT to 30 minutes to reduce the effect of heat buildup on the final results. In this case, the charge passed after 30 minutes is simply multiplied by 12 to obtain the total charge passed for the RCPT.

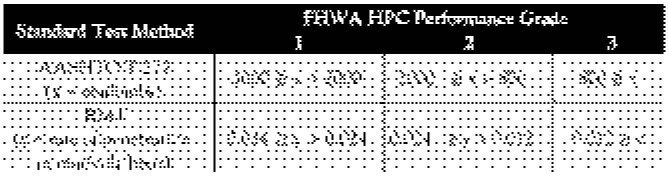
$$D_{eff} = 1.03 \times 10^{-14} (Coulombs)^{0.84} \quad (2\text{-}3)$$

$$D_{eff} = 0.88 \times 10^{-14} (Coulombs)^{0.76} \quad (2\text{-}4)$$

where:

D_{eff} = Effective diffusion coefficient.

The RMT and bulk diffusion tests both supply diffusion coefficients for a given concrete mix. These coefficients are often used to relate the data from these two tests together. Also, the rate of penetration of the chloride in the RMT test, given in in. per volt-hour (mm/volt-hour) is also used (Hooton et al., 2001). Hooton et al. suggest using the information in Figure 2-3 to compare the RCPT results to the RMT results, and to rate the concrete mixes.



Standard Test Method	FHWA HPC Performance Grade		
	1	2	3
AASHTO T 277 (X = coulombs)	3000 ≥ x > 2000	2000 ≥ x > 800	800 ≥ x
RMT (X = rate of penetration in in./volt-hour)	0.046 ≥ x > 0.024	0.024 ≥ x > 0.012	0.012 ≥ x

Figure 2-3. HPC chloride penetration resistance performance grades (from Hooton et al., 2001) (25.4 mm = 1 in.)

They also recommend using the rate of penetration from the RMT results to compare to the salt ponding test and the bulk diffusion results. For comparison between the ponding test and RMT, the chloride content in the upper 0.5" of the ponded specimen was used for the correlation and the r^2 value was 0.735, while the correlation with the diffusion coefficient from the bulk diffusion test was better, having an r^2 of 0.865. During the same project, the RCPT was also compared to the ponding test and bulk diffusion. The results from this correlation were not quite as good as the RMT provided, indicating that the RMT might provide a better representation of a concrete's chloride penetrability.

2.7 Chloride Profiles in Concrete

The concentration of chloride at increasing depths in a concrete profile is dependent on factors besides the material characteristics of the concrete. In addition to the diffusion coefficient that governs this sort of transport through the concrete, non-diffusion ingress is permitted by cracks. That cracks occur during the service life of concrete structures is not unusual, and are the results of shrinkage, creep or service load conditions. However, the reduction in durability due to chloride vulnerability of cracked concrete is considerable. The concentration of chloride ions at depths near the reinforcing steel can be significantly higher than required for corrosion initiation (West and Hime, 1985).

Diffusion profiles in concrete produce concave up curves similar to the one shown in Figure 2-4. In the flatter portion of the curve it is assumed that the chloride levels are approaching the baseline, or initial chloride content of the concrete.

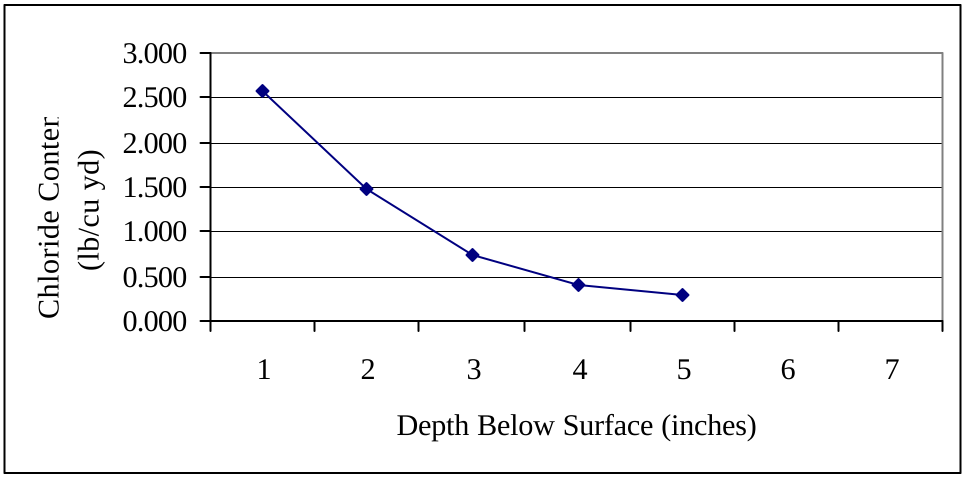


Figure 2-4. Typical chloride content profile

Figure 2-5 illustrates the chloride content of an area on the abutment of a bridge in Raleigh, NC. There was evidence that drainage from behind the abutment washed over the area where the sample was taken. The profile departs from the expected diffusion curve because the chloride is dissolved from the upper layers of the concrete as water moves over it.

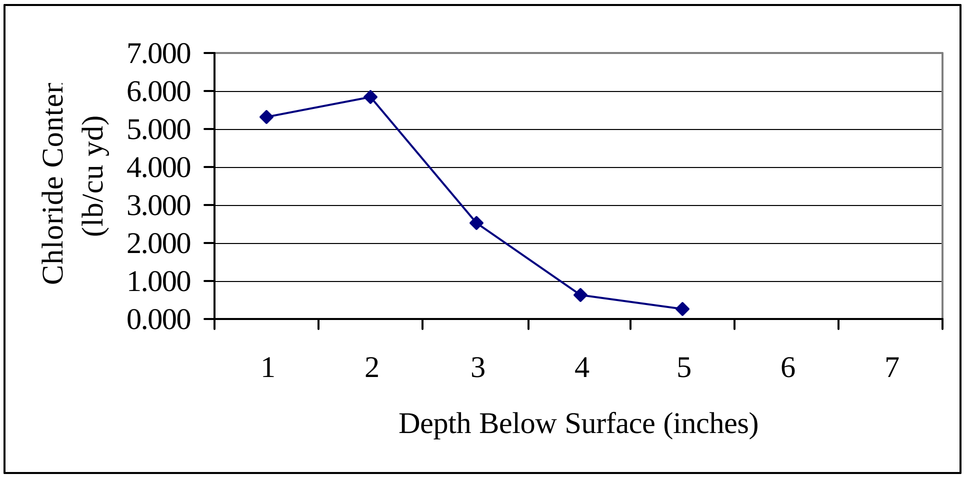


Figure 2-5. Chloride profile of concrete in service

Near the surface of the concrete, processes besides pure diffusion are at work, and as was just illustrated in the abutment example, there is more interaction with the outside environment. In-service exposures to conditions that affect the surface such as rainfall, street washing, and surface abrasion by vehicular traffic affect the shape and magnitude of the chloride profile. In addition, the condition of the surface is predetermined during construction by concrete finishing processes. Consequently, this surface region is not ideal for inclusion in a diffusion model. However, the surface region does provide the reservoir of chloride that will eventually penetrate to more critical levels near the steel reinforcement (West and Hime, 1985).

An FHWA study suggests the use of a model boundary surface beneath the actual surface of the concrete (Weyers et al., 1994). It has been noted that the concentration of chloride ions comes to a semi-stable state after approximately four years of chloride exposure. Although the concentration is not perfectly stable, its fluctuation is not nearly as radical as that of the actual concrete surface, which is subject to washing, rain and intermittent chloride application (Weyers et al., 1994).

2.8 Corrosion Threshold

From a design and maintenance perspective, information on chloride content is only useful in relation to a concentration that will initiate destructive processes in the reinforcing steel. This value is commonly called the corrosion threshold, and describes the chloride content at which steel becomes de-passivated, and it is likely to begin corroding. Because the corrosion reactions are fairly complex, and are affected by both the steel properties as well as the chemical composition of the concrete pore solution, exacting corrosion thresholds can only be stated after having been determined experimentally for specific concrete mix designs and their steel reinforcement.

Tests to determine this threshold include ASTM C109-92, “Standard Test Method for Determining the Effects of Chemical Admixtures on the Corrosion of Embedded Steel Reinforcement in Concrete Exposed to Chloride Environments” (ASTM, 1992). Other test methods have been devised by individual researchers. General thresholds may be stated; however, these are estimates at best.

The threshold is most simply stated as chloride content in units of a percent by mass of cement. Many bridge maintenance documents use this format. This number is obviously most accurate when the cement content of the concrete is known. Older concrete components might have unknown mix designs and their cement content must be approximated in order to state their chloride content as a percentage of the cement mass. This further reduces the accuracy of comparing corrosion threshold values with actual, field measured values. The steel becomes vulnerable to corrosion only after the pore solution pH is reduced to levels below 10, or when chloride levels reach the corrosion threshold. Because of the cement’s chemical composition, there is a considerable amount of alkalinity to the concrete pore solution. Therefore, the amount of cement in the concrete is an important indicator of the rate that the pH will be lowered to critical levels.

However, there are many other methods of stating the threshold that sometimes appear more accurate in predicting the chloride content associated with corrosion initiation. One commonly stated threshold is related to the Cl/OH ratio. The ratio is thought to affect the probability that the steel will react with the hydroxyl ion to cause passivation or to react with the chloride ion to initiate corrosion. An abundance of studies have found critical Cl/OH ratios that are dissimilar. The impracticality of using the ratio is that it is very different for specific concrete mixtures. The use of this ratio without careful testing of its validity on the concrete of interest will produce erroneous service life predictions (Thomas et al., 1995). Concerning the concretes studied in this project, the fly ash content will present a challenge in estimating the corrosion threshold. It is known that fly ash tends to reduce the threshold value as more cement is replaced. A study by Thomas et al. (1995) found the following relationship between fly ash levels and corrosion thresholds, shown in Table 2-2.

Table 2-2. Corrosion threshold changes with addition of fly-ash

Fly Ash (% cement replacement)	% Chloride Threshold (by mass of cementitious material)
0	0.7
15	0.65
30	0.5
50	0.2

Although the corrosion threshold decreased with increasing fly ash replacement levels, the quality of the concrete can still be considered higher. Thomas found that the fly ash was effective at slowing the ingress of chloride ions to the depth of the steel. There were significantly reduced chloride levels deep in the concrete profile of samples containing fly ash (Thomas et al., 1995). Therefore, it is essential to consider the reduced threshold simultaneously with decreased chloride permeability, in order to model the service life and deterioration rate of concrete structures.

For purposes of this study, the chloride threshold used was that proposed by the FHWA for bridge assessment (Weyers et al., 1994). The level considered as the corrosion threshold ranges between 1.2 lb/yd^3 and 1.7 lb/yd^3 depending on the proportion of chloride that is water soluble. In some NCDOT documents the basic corrosion threshold used (i.e. no experimental process has been used to determine a corrosion threshold specific to the concrete under investigation) is 1.4 lb/yd^3 (Rochelle, 2001). Therefore, in the present study the corrosion threshold used was 1.4 lb/yd^3.

2.9 Design Goals for NC Bridges

The current design policy for structures situated in corrosive environments consists of a prescriptive specification for the addition of admixtures and secondary cementitious materials. This policy was put forth in a memorandum to Project Engineers and Project Design Engineers at the NCDOT on February 29, 2000. It specifies four regions that should receive special treatment for their exposure to chloride. These are the two coastal regions shown in Figure 2-6 and two inland regions.

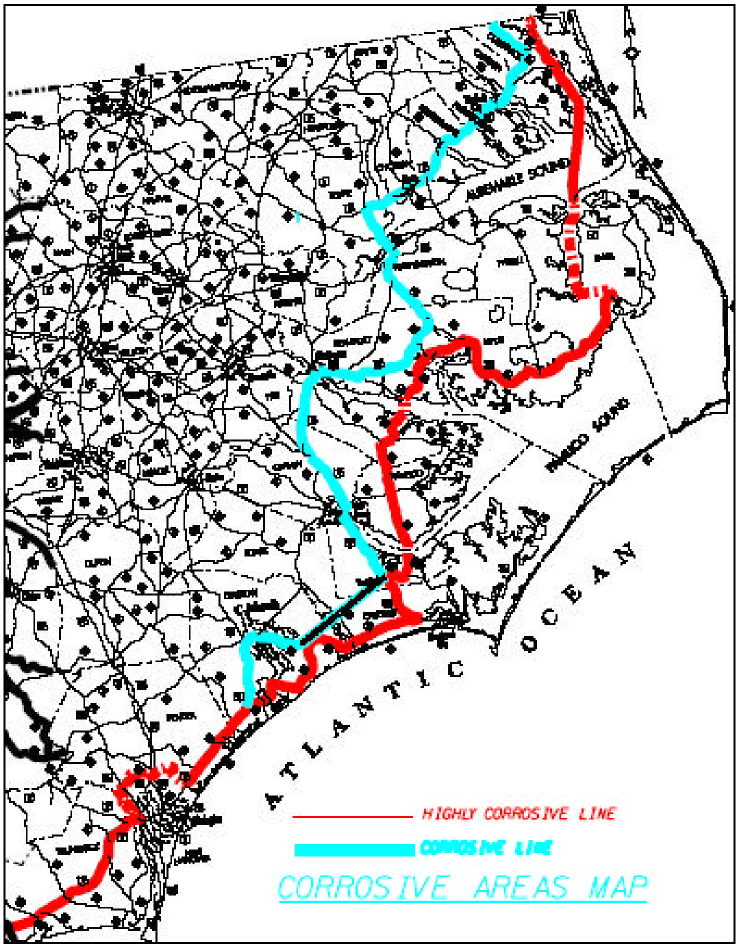


Figure 2-6. Corrosive areas defined for coastal areas of North Carolina

The outer region shown on the map is considered highly corrosive and the inner region is considered corrosive. The text of the memo specifies:

- For structures located in the highly corrosive area of the state, including those structures in corrosive areas with any portion located less than 15 feet above mean high tide, substructure cast-in-place concrete shall contain 2.0 gal/yd^3 of calcium nitrite corrosion inhibitor.
- For those structures located in the highly corrosive area of the State, 5% of the Portland cement shall be replaced with silica fume in those elements of the structure that may undergo repeated wetting and drying cycles.
- For structures in divisions 5, 7 and 9-14 (see Figure 2-7), the bridge deck shall contain fly ash or ground granulated blast furnace slag.
- Due to concerns regarding the potential for galvanic cell corrosion of un-coated prestressing strands in the presence of epoxy coated mild reinforcing steel, the mild reinforcing steel in all prestressed piles and girders shall no longer be epoxy coated.
- As a clarification of current policy, in highly corrosive areas, or corrosive areas where any portion of a cast-in-place concrete member is less than 15 ft above mean high tide, all bar supports in that member and all like members shall be epoxy coated.

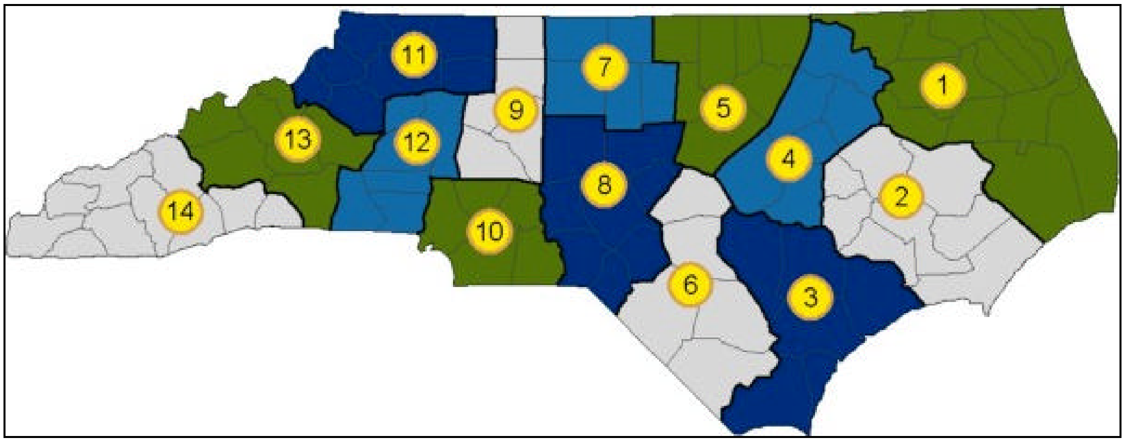


Figure 2-7. NCDOT maintenance divisions

2.10 Service Life Modeling

In light of chloride ingress modeling techniques, it should be possible to create a more specific corrosion policy for the coast as well as for other regions of North Carolina. Rochelle (2001) put forth the modeling technique that was used to design the

Virginia Dare Bridge in Manteo, NC. The model uses the traditional Fickian diffusion formulas to predict the maintenance free service life, T_{corr}, of bridges in aggressive chloride environments. Concrete mix designs were selected to meet chloride permeability limits selected through use of the model.

As it has been described in the preceding sections, corrosion in concrete is initiated when a specific concentration of chloride reaches the depth of the reinforcing steel. Once the corrosion process is initiated, the concrete tends to crack and spall, eventually limiting the serviceability and safety of the structure. The endpoint of diffusion based model is the moment when the concentration of chloride equal to the corrosion threshold is reached at the depth of the steel. From the time this accumulation of chloride is reached, it is assumed that major maintenance will be required within five years. Therefore, a 100 year service life projection requires a model that shows an accumulation of chloride sufficient to initiate corrosion occurring at 95 years.

Given this summary of service life modeling, it follows that several quantities are required to accurately predict the chloride ingress process. The corrosion threshold, the rate at which salt is applied to the surface (or the constant surface concentration for marine bridges), the diffusion characteristics of the concrete and the length of time in service are all terms in the model formulas.

Determination of the conditions that will initiate corrosion requires testing, assumption or heavy research. Many techniques are used to reduce the susceptibility of the steel to chloride attack. These include addition of corrosion inhibiting admixtures, such as calcium nitrite, or physically protecting the bars with epoxy coating. The addition of corrosion inhibiting chemicals is related to the model by increasing the corrosion threshold by an appropriate chloride concentration.

For instance, in the model used by NCDOT, the corrosion threshold for concrete without admixtures or physical protection of the reinforcing bars is assumed to be 1.4 lb/yd^3. Addition of 2 gal/yd^3 of calcium nitrite increased the corrosion threshold to 6.0 lb/yd^3. The use of epoxy coated reinforcing steel was related to the model by laterally shifting the chloride concentration curve to add service life. The amount of lateral shift was 8-10 years. It is important to note that some of the admixtures that are used in concrete can have the affect of lowering the corrosion threshold. An example of this is the addition of fly ash, which was discussed earlier.

The surface concentration of chloride is an essential parameter for the model. Bridge elements are individually subjected to different surface concentrations based on their proximity to the source of the chloride, and their exposure to processes that remove chloride from the surface such as rain or street washing. In some bridge elements, the surface concentration is immediately at a stable maximum. Instances of this loading regimen would be as in a footing where the surface is regularly wetted by water with a consistent salinity.

In some cases, the best model of surface concentration allows for the gradual build up of chloride over time rather than an instant maximum. Bridge decks, columns and other elements that receive their chloride loading intermittently or cyclically are examples of such a condition.

Table 2-3 illustrates the input parameters for the diffusion model of the deck slab, the columns and the pile caps for the Virginia Dare Bridge (Rochelle, 2001). It can be seen that the surface chloride concentration is adjusted for the expected exposure of elements to chloride sources. The k, or surface concentration build-up coefficient, applies only to the deck slab and the columns. The pile caps are assumed to have constant contact with the salt water in the bay. The diffusion coefficients represent a range of reasonable concrete characteristics.

Table 2-3. Input parameters for NCDOT chloride ingress model

	D (in^2/year)	C_0 (lb/yd^3)	k (lb/yd^3 *yr)
Deck Slab	0.0783-0.147	5.1	0.51
Columns	0.0783-0.147	9.9	3.03
Pile Caps	0.0783-0.147	19.0	-

The values in Table 2-3 were derived from a number of sources. D, the diffusion coefficient is partially a function of cement content. The w/c ratio for North Carolina concretes is no more than 0.40 - 0.426 (based on concrete class). In general, diffusion coefficients for this range of w/c ratios are between 0.07 and 0.15 in^2/year. In addition, the NCDOT performed an analysis of existing coastal structures which found diffusion coefficients which ranged from 0.06 to 0.12 in^2/year (Rochelle, 2001).

To specify material characteristics for the Virginia Dare Bridge, hundreds of permutations with the input parameters in Table 2-3 were modeled. Each permutation consisted of a single combination of characteristics from a set of five doses of calcium nitrite, five chloride loading scenarios and six diffusion coefficients (Rochelle, 2001).

After extensive material testing and durability modeling, a prescriptive specification was developed for each type of structural element in the bridge. The specification included two calcium nitrite dosages and three different combinations of fly ash and microsilica inclusion rates for various elements.

To expand the application of the design methodology used for the Virginia Dare Bridge, it was desired to compile exposure information from other regions of the state. By sampling the concrete from bridges in various areas of the State, it will be possible to determine their exposure to chloride and to select a mitigation strategy to be used in the design of future bridges. Once a large enough selection of concrete mix designs have been tested for their diffusion characteristics, the durability prescription can be used as routinely as strength prescriptions.

3. PROCEDURES

3.1 Concrete Mix Designs

The 10 concrete mixes were selected by NCDOT officials to represent a wide range of concrete mixes currently in use in North Carolina (summary shown Table 3-1). These concrete mixes were either Class A or Class AA mixes, which are specified for their suitability in various bridge elements. There was also a Class AA lightweight bridge deck mixture tested. This mixture was selected to determine how susceptible lightweight concrete is to chloride penetration. Every mix design, as well as the fresh concrete test results obtained during the mixing can be seen in Appendix A.

Both the AASHTO and ASTM test methods require the creation of concrete slabs for the ponding test. These specimens were created using wooden forms that permitted production of 4 slabs (see Figure 3-1), each measuring 12 inches square by three inches thick. The forms were sprayed with an oil-based form release prior to the addition of the fresh concrete to ensure easy removal of the slabs. The mixing and curing of the concrete was completed in accordance with ASTM C192 (ASTM 1990).

After the concrete was mixed, it was placed into the wooden form for 24 hours. After 24 hours, the slabs were demolded and moved to a wet curing room maintained at 100% humidity and 73° F temperature. After 14 days the slabs were moved from the wet curing room to an environmental chamber with 50±4% humidity and 73±3°F, in accordance with AASHTO T259. To ensure that the humidity and temperature were kept constant, they were measured continuously using a HOBO temperature and humidity reader. When the slabs were 29 days old, Plexiglas dams were attached to the top surface using silicon caulking. The slabs were then ponded with a 3% NaCl solution after the dams had been in place for an additional 13 days. Once the solution was added, the salinity was monitored on a weekly basis using a Salintest device manufactured by Hanna Instruments. The solution was adjusted regularly based on the readings obtained.

Table 3-1. Summary of mix designs considered

MIX NO.	MIX TYPE	CEMENT (lbs/cy)	POZZ. (lbs/cy)	FA (lbs/cy)	CA (lbs/cy)	WATER (lbs/cy)	AIR (gal/cy)	RET. (gal/cy)	WATER RED. (gal/cy)	SUPER PLAST. (gal/cy)	CORR. INHIB. (gal/cy)	SILICA FUME (lb/cy)	MEAS. AIR (%)	MEAS. SLUMP (in.)
1	AA DECK	564	170	1288	1375	286	0.014	0.114	0.022	-	-	-	2.5	6.5
2	AA	677	-	1116	1900	276	0.022	0.022	0.057	-	-	-	2.0	6.5
3	A	678	-	1018	1901	294	0.108	0.108	0.095	-	-	-	4.0	6.0
4	AA FTG.*	448	208	1110	1755	230	0.071	0.143	-	0.927	3.003	35.0	5.5	6.0
5	AA COLS.*	556	148	1122	1617	247	0.200	0.143	-	1.178	3.002	37.8	7.0	7.8
6	A FTG.	451	136	1153	1570	289	0.120	0.016	0.057	-	-	-	5.0	2.5
7	PRTR. GRD.*	751	-	1055	2040	208	0.011	-	0.088	0.365	2.501	-	4.5	7.3
8	PRTR. GRD.*	519	139	1042	1903	208	0.057	-	-	0.479	3.601	-	6.0	7.0
9	DECK	572	172	1023	1900	267	0.109	0.232	NA	-	-	-	4.5	2.5
10	AA DECK LW/FA	715	-	1234	900	294	0.036	0.107	0.201	-	-	-	8.5	2.5

For each concrete mix, the slump and the air content were measured in accordance with the respective ASTM standards. The air content of the concrete mixes was also specified in the mix design, and was measured by either the pressure method or the volumetric method. The volumetric method (shown in Figure 3-1) for the determination of air content was used for mix 10, which contained lightweight coarse aggregate. The pressure method was used for the remaining mix designs. There was a mistake made in the determination of the air content for mixes 1 and 7, as the volumetric method should have been used, due to the fact that the coarse aggregates in these mixes had high porosities. A method for correcting these measurements will be presented later.

Figure 3-1. Measuring the air content by the volumetric method

There were a total of 10 mix designs that were tested for this project. All of these concretes were created in UNCC's materials lab with the exception of mix 9, which was collected at a bridge construction site as the deck was being poured. The final trial batch for each design was approximately 2.5 ft^3 in order to ensure that there was enough concrete to create four 12"x12"x3" slabs and a minimum of four 8" cylinders, as well as to measure the air content and slump.

There were two slabs made for the first trial batch of each mix except for mixes 1 and 6. One of these slabs was ponded and the other was used as the control. There were also two cylinders made from every trial batch, which were used for RCPT tests (see

Figure 3-2). These additional slabs and cylinders were made in order to investigate the effect air content and slump had on the RCPT and ponding test results (as these trials were identical to the final batch for each mix, except for the additives influencing slump and air content).

Figure 3-2. Ponding slab forms and cylinders

Some of the admixtures and aggregates specified in the mix design were no longer available. In these cases, a representative of the admixture manufacturer was consulted for a suitable alternative. The Materials and Tests Unit at the NCDOT specified replacement aggregates when the original quarry closed for business. These substitutes, as well as the concrete mix results can be seen in Appendix A.

3.2 RCPT

The Rapid Chloride Permeability Test is performed by applying a 60 V_{DC} charge to a 4” concrete cylinder that is 2” thick. The cylinder was exposed to a 3% Sodium Chloride, NaCl, solution on one side, and a 0.3 N Sodium Hydroxide, NaOH, on the other. These solutions were contained in cells made out of acrylic plastic.

For this project, the cylinders that were tested with the RCPT machine were cured in the same conditions as the slabs that were being ponded. This meant that there were four mixes that were tested after six months, and then all the cylinders tested after one year. The four mixes that were tested after six months were the only mixes that did not contain supplementary cementitious materials such as silica fume and fly ash. NCDOT requested from the research group to determine if a shorter test period could be used with these types of concrete mixes.

AASHTO T277 states that the test shall run for six hours, with measurements of the coulombs passed and the current taken every thirty minutes (AASHTO, 2002). The only exception to this was when the coulombs passed began to run high. The test was discontinued when the coulomb count reached 6000, or when the temperature of the solutions in the cells reached 185^{o}F (85°C). This was done to prevent damage to the cells and the electrodes in the cells. This did not affect the conclusions from the RCPT results because any reading over 4000 coulombs is considered very highly permeable concrete based on AASHTO T277 (2002) and ASTM C1202-02 (ASTM, 2002).

3.3 Salt Ponding Test

The main procedure guiding the ponding test is AASHTO T 259, “Standard Test Method for Resistance of Concrete to Chloride Ion Penetration” (AASHTO 2002). This test procedure provides the guidelines used for the ponding test and is widely used by DOTs across the country. There were a few deviations made from the procedure to accommodate specifics of this particular project. The AASHTO test method is written specifically for a 90-day ponding period. This duration was deemed insufficient for some of the low permeability concretes under investigation in this study. Changes to the AASHTO method were made after referring to ASTM C 1543, which has considerations for longer ponding periods.

Since these tests were being conducted for longer than ninety days, the salt solution was changed every two months, as recommended in ASTM C 1543 (ASTM 2003), and the solution salinity was checked every week. Samples without pozzolans or corrosion inhibitors were tested initially after 6 months of ponding, as well as after one year. Samples taken from the specimens were tested following the AASHTO T 260 procedures for acid soluble chloride content in concrete.

There were a few problems encountered with slab ponding. When the environmental chamber was first installed it had difficulties keeping constant humidity and temperature because the wrong control unit was originally installed by the manufacturer. This problem was solved after modifications were made by technicians from the manufacturer of the conditioning unit. There have been no further problems with the chamber's control unit since the modifications.

The most persistent problem has been the seal of the Plexiglas dams to the slabs. This was initially done by spreading silicon rubber caulking on the dam, and then applying it to the slab. A small amount of silicon was used at first, but this allowed the dams to leak after the solution was added. To compensate for this, more silicon was used and the dams were resealed after the initial silicon had dried. This eliminated most of the leaking problems with only a few exceptions.

The final challenge was related to covering the dams and maintaining the appropriate salinity of the ponding solution. Initially, plastic food wrap was used to cover the ponded surface and prevent evaporation. This was not an effective way to cover the slabs because the plastic wrap was too elastic and would sag into the solution. This problem was solved by replacing the plastic wrap with heavier sheets of plastic that were taped to the sides of the dams (see Figure 3-3).

Figure 3-3. Ponded slabs in environmental chamber

3.4 Potentiometric Titration

Chloride concentration in concrete is commonly tested with one of two test procedures. The first is ASTM C114, and the second is AASHTO T260. There are significant differences to the sample preparation process between these two procedures; however, in both the chloride concentration is determined by a potentiometric titration. The titration is a lengthy procedure, which requires a laboratory setting, specialized equipment and trained personnel. During this project AASHTO T260 was used to determine the chloride content of the concrete by titration.

The AASHTO procedure begins with the collection of a sample of concrete powder that is sufficiently fine to pass a #50 sieve. The powder is heated in a strong acid solution (HNO_3), which digests the solid particles and releases the chloride ions bound to the concrete. The remaining solid material is removed by a #41 Whatman paper laid over a #40 Whatman paper. The filtered solution is then allowed to cool, and is finally titrated with a 0.01 N $AgNO_3$ solution.

The potentiometric titration process proceeds as follows. An electrode sensitive to Chloride (Cl^-) ions is immersed in the filtered solution. As the $AgNO_3$ titrant is added to the filtered solution containing the chloride ions, the electrode registers an electrical potential in mV. As the Ag^+ ions react with the Cl^- ions a precipitate is formed, removing the Cl^- ions from the solution and raising the potential registered by the electrode. Titrant is added incrementally and a mV reading is recorded at each step. The condition of the concentration of Ag^+ ions being equal to the concentration of Cl^- ions is known as the equivalence point (Willard 1981). Near the equivalence point, equal additions of the titrant cause increasingly large changes in potential as the number of Ag^+ ions becomes similar to the number of Cl^- ions.

In the lab, the equivalence point is determined by plotting a graph of the change in mV per addition of $AgNO_3$, as is shown in Figure 3-4. The equivalence point is near the $AgNO_3$ addition that causes the largest change in potential. The quantity of $AgNO_3$ required to reach the equivalence point can be used directly to calculate the concentration of Cl^- ions that were reacted during the titration. At UNCC the test operators used standard data sheets for each individual titration (Bledsoe, 2005).

Quantity of titrant added

Change in mV per addition of $AgNO_3$

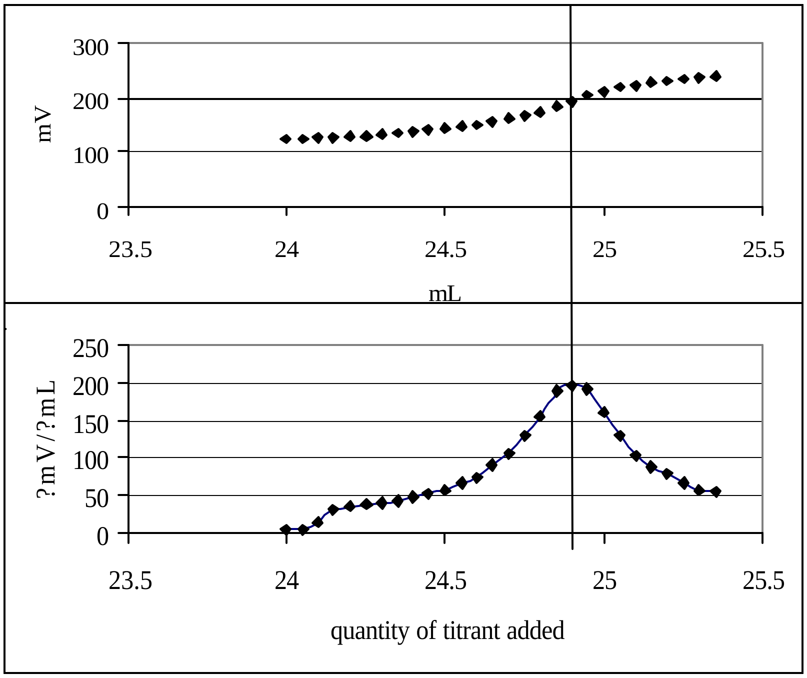


Figure 3-4. Typical titration results (1 mL = $2.64*10^{-4}$ gal)

3.5 RCT

In 1993, the FHWA published an evaluation of several potential rapid chloride testing methods (Strategic Highway Research Program, 1993). The method, which was deemed most economical and reliable, uses an electrode sensitive to Cl^- ions to measure the concentration of Cl in a digestion solution, developed to extract chloride ions from the concrete powder. The test procedure is referred to as the Rapid Chloride Test (RCT), and the complete testing kits are available from several manufactures. To evaluate the rapid testing method, a kit from Germann Instruments was purchased. The Germann testing kit included equipment for collecting powder concrete samples, a specific ion electrode, a mV meter, calibration solutions and digestion solutions (see Figure 3-5).

Figure 3-5. Rapid chloride testing device

The process of testing concrete powders begins with calibration of the mV meter and electrode (shown in Figure 3-6). Calibration solutions, prepared by the manufacturer, have chloride ion concentrations of 0.005%, 0.020%, 0.050% and 0.500%, or .190, .763, 1.91, and 19.1 lb/yd^3, respectively. This conversion was made using Equation 3-1. The electrode is immersed in each solution and the potential in mV corresponding to each chloride concentration is plotted on a semi-log graph paper. A straight line is fitted between the four points. Alternatively, the four calibration points are entered into a spreadsheet application and a linear regression is performed to generate an equation that relates electrical potential to chloride concentration. Figure 3-7 shows an example of a regression line drawn in Microsoft Excel using the mV readings obtained during calibration.

$$W_{Cl} = 38.15 * (\%Cl) \qquad (3\text{-}1)$$

where:

W_{Cl} = Weight of chloride in lb/yd^3;

%Cl = Percent chloride from RCT reading.

Figure 3-6. mV meter and specific ion probe

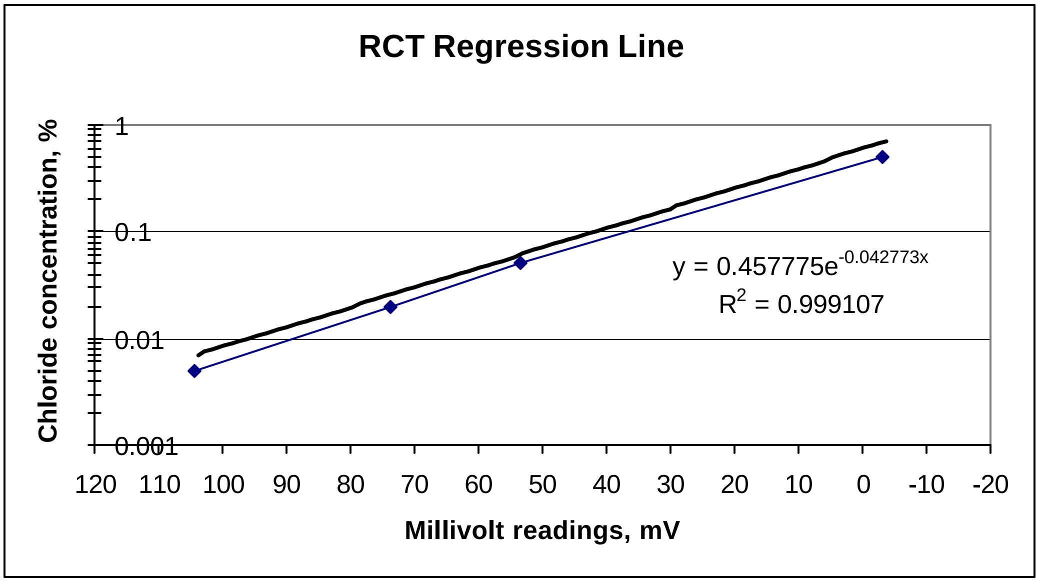


Figure 3-7. Sample RCT regression line for determining chloride content

The powder samples are generated in the same fashion as the samples used in the AASHTO titration. A 0.0033 lbs (1.5 gram) sample is added to a vial of the proprietary extraction solution. After the vial has been shaken for five minutes, the ion-specific electrode is inserted into the solution and a potential in mV is registered on the voltmeter.

The reading from the voltmeter is related to chloride content by way of the regression line found in the calibration process. In order to correlate these data with full digestion results, the values were then entered into one of two equations. If the chloride content was greater than, or equal to 0.010%, Equation 3.2 was used. If the content was between 0.003% and 0.010%, Equation 3.3 was used. For values equal to and below 0.003%, no correction factor was used (Tempest, 2004).

$$\%Cl = 1.16(RCT) + 0.0077 \qquad (3.2)$$

$$\%Cl = 1.0949(RCT) + 0.0059 \qquad (3.3)$$

One aspect of the NCDOT project was to evaluate the RCT equipment and compare its results to those from the AASHTO titration. Brett Tempest compiled the results from these two tests and performed a linear regression analysis to determine the accuracy of the rapid method. The graph in Figure 3-14 shows the results of this analysis (Tempest, 2004), proving very good correlation between the two methods. Based on these result, and approval from the NCDOT TAC committee, the RCT was used for the determination of the chloride content in the concrete (Tempest, 2004). For further quality control, titrations were completed for 10% of the remaining samples.

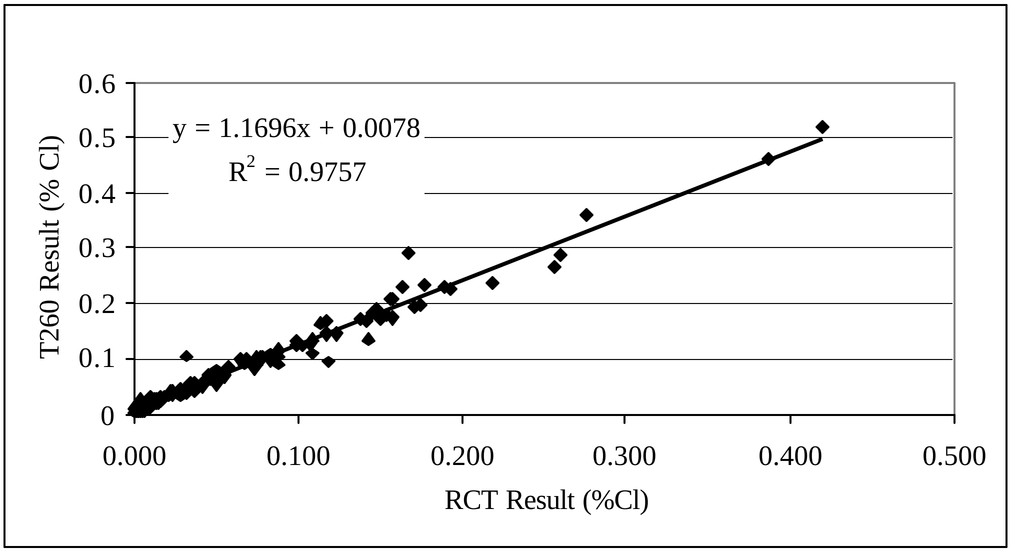


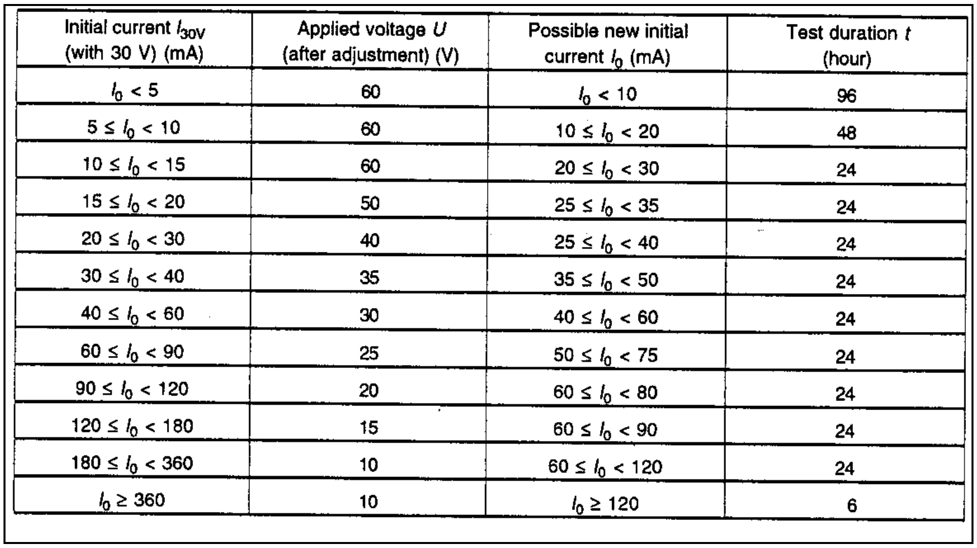
Figure 3-8. Relationship between RCT and titration results, from Tempest (2004)

3.6 RMT

The rapid migration test (RMT) was developed by Tang and Nilsson in 1991 and was later adopted as Nordtest NT Build 492 (Nordtest, 1999). Nordtest is the ASTM equivalent in the Nordic region in Europe. This test has not been officially adopted by any American testing agencies as of yet, but it is undergoing review by an ASTM committee (Hooton et al., 2001). The test method is used to determine the chloride migration coefficient of a concrete mix from a non-steady-state migration experiment.

The procedure for the RMT is similar to that for the RCPT, with several key differences. The specimen is a 4” diameter concrete cylinder that is 2” thick. The specimen is vacuum saturated for three hours, and then it is covered with a saturated calcium hydroxide, $Ca(OH)_2$, for an additional hour. The specimen is then left submerged in the $Ca(OH)_2$ solution for 18 ±2 hours (Nordtest, 1999). After the sample is removed from the $Ca(OH)_2$, it is siliconed into a plastic cylinder mold. Once the silicon cures, the sample is placed into a reservoir containing 12 liters of 10% NaCl solution. The plastic cylinder is then filled with 0.079 gal (300 mL) of 0.3 N NaOH solution. 30 V_{DC} are then applied to the specimen, and the initial current is recorded. Based on the initial current, the test voltage is set and the test started. The test voltages and times can be seen in Table 3-2 (Nordtest, 1999).

Table 3-2. Test voltages and duration for RMT (Nordtest, 1999)



Initial current I_{30V} (with 30 V) (mA)	Applied voltage U (after adjustment) (V)	Possible new initial current I_0 (mA)	Test duration t (hour)
$I_0 < 5$	60	$I_0 < 10$	96
$5 \le I_0 < 10$	60	$10 \le I_0 < 20$	48
$10 \le I_0 < 15$	60	$20 \le I_0 < 30$	24
$15 \le I_0 < 20$	50	$25 \le I_0 < 35$	24
$20 \le I_0 < 30$	40	$25 \le I_0 < 40$	24
$30 \le I_0 < 40$	35	$35 \le I_0 < 50$	24
$40 \le I_0 < 60$	30	$40 \le I_0 < 60$	24
$60 \le I_0 < 90$	25	$50 \le I_0 < 75$	24
$90 \le I_0 < 120$	20	$60 \le I_0 < 80$	24
$120 \le I_0 < 180$	15	$60 \le I_0 < 90$	24
$180 \le I_0 < 360$	10	$60 \le I_0 < 120$	24
$I_0 \ge 360$	10	$I_0 \ge 120$	6

The RMT was not part of the original the project (it was considered later to investigate the effectiveness of RMT to quantify chloride diffusion), so the only mixes that were tested with the RMT were 7 through 10. Prior to conducting the tests, the cathode and anode had to be manufactured in-house. The cathode, which was submerged in the NaCl solution, was constructed from a stainless steel plate 0.02” (0.5 mm) thick. The test method did not give what dimensions the cathode should be, so it was constructed to be approximately the size of the concrete cylinder, 4” in diameter.

The anode was constructed from a 0.02” (0.5 mm) thick stainless steel mesh, and it was submerged in the NaOH solution. The anode was constructed so that it could fit inside the plastic cylinder. A picture of the cathode and anode can be seen in Figure 3-9. The container for the NaCl solution was a plastic tank with a 5.28 gal (20 L) capacity. The cylinder was supported on a plastic support that was cut to 32°, as required by the standard (Nordtest, 1999). The reservoir with the support and the entire test setup can be seen in Figure 3-10.

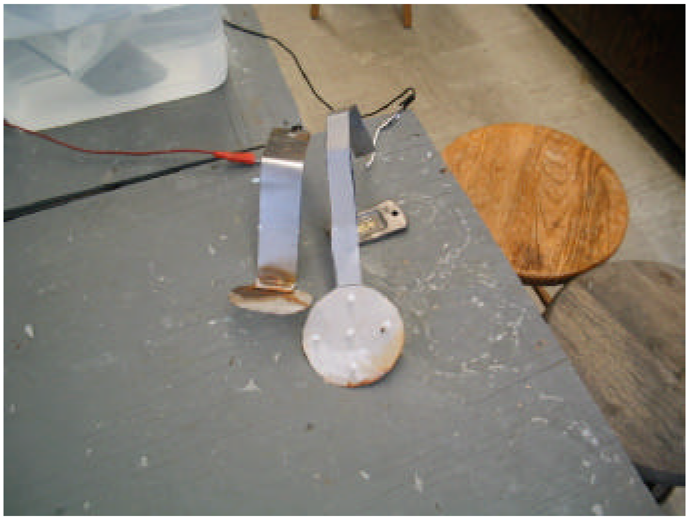


Figure 3-9. Anode (left) and cathode (right) for RMT

Figure 3-10. RMT test set-up

After the test ran for the time specified in Table 3-2, the test was ended and the specimen was removed from the plastic tank. It was then split using a hammer and chisel. The split surface was first measured with digital calipers at four locations to determine the average thickness, and then sprayed with a 0.1 molar silver nitrate solution. The silver nitrate solution reacts with the chloride present to form a white precipitate. After the reaction was complete (approximately 15 minutes), the depth of chloride penetration was measured using digital calipers. The measurements were made based on the method outlined in the standard. There were a minimum of 7 measurements made, all 0.39” (10 mm) apart. The outer 0.39” (10 mm) on either side of the cylinder was ignored (Nordtest, 1999). The measurements made were then placed into an equation to determine the non-steady-state diffusion coefficient of the concrete. The split cylinder before and after the application of the silver nitrate solution can be seen in Figures 3-11 and 3-12.

Figure 3-11. Split cylinder prior to silver nitrate application

Figure 3-12. Split cylinder before and after silver nitrate application

Once all of the data was collected from the RMT test, it was entered into an Excel spreadsheet and the effective migration coefficient was calculated based on Equation 3-4. An example of this worksheet can be seen in Figure 3-13.

$$D_{nssm} = \frac{0.0239(273+T)L}{(U-2)t}\left(x_d - 0.0238\sqrt{\frac{(273+T)Lx_d}{(U-2)}}\right) \qquad (3\text{-}4)$$

where:

D_{nssm}: non-steady-state migration coefficient, $x10^{-12}$ m^2/s (1.0 $x10^{-12}$ m^2/sec = 0.0487 $in^2/year$);

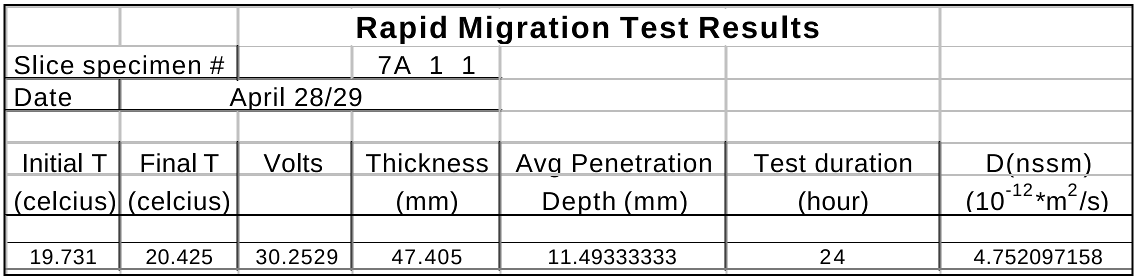
U: absolute value of the applied voltage, V;

T: average value of the initial and final temperatures in the anolyte solution, °C;

L: thickness of the specimen, mm (25.4 mm = 1”);

x_d: average value of the penetration depths, mm (25.4 mm = 1”);

t: test duration, hours.



		Rapid Migration Test Results				
Slice specimen #			7A 1 1			
Date	April 28/29					
Initial T (celcius)	Final T (celcius)	Volts	Thickness (mm)	Avg Penetration Depth (mm)	Test duration (hour)	D(nssm) $(10^{-12}*m^2/s)$
19.731	20.425	30.2529	47.405	11.49333333	24	4.752097158

Figure 3-13. Example of migration coefficient calculation in Excel©

3.7 Bulk Diffusion Test

The bulk diffusion test is another long-term test for determining the chloride permeability of a concrete mix. It was first introduced as a Nordtest Method similarly to the RMT, but was adopted as an ASTM standard in 2003 (ASTM, 2003). This test method is performed on 4” diameter concrete cylinders that are prepared according to ASTM C1556.

The test specimen is obtained from a 4” cylinder that has been cured in accordance with ASTM C31 by slicing the top 3” of the cylinder in a wet concrete saw (ASTM, 2003). Also, a 0.79” thick slice is cut from the remaining portion of the cylinder to be crushed and sampled for the background chloride content. The test specimen is then placed into a room with 50% relative humidity and 73°F for 24 hours. This is to ensure that the surface of the concrete is dry, but it is still internally moist (ASTM, 2003). All sides of the specimen are then sealed with epoxy, save for the top finished surface. Once the epoxy has cured, the initial mass of the specimen is determined and then it is submerged in a calcium hydroxide bath. The specimen is removed and its mass recorded every 24 hours until the percent change in mass is less than or equal to 0.1% (ASTM, 2003). Once this is met, the sample is placed into the exposure solution, which is an aqueous NaCl solution containing 1.78 lbs of NaCl per gallon of solution (165 gram/Liter). The minimum exposure time is 35 days, but for high performance concretes, this time should be increased.

As was the case for the RMT, only four mix designs were tested with the bulk diffusion procedure. Once the samples were removed from the exposure solution, they were allowed to dry overnight in the environmental chamber. If they were not able to be sampled right away, they were sealed in plastic bags to prevent the further diffusion of chloride ions. The cylinders were placed into a milling machine equipped with a 1 1/8" diamond-tipped core bit. The drill rotation was set to 320 rpm and the cross-bed travel speed to 2" per minute, based on recommendations from Hooton and McGrath (1999). The sampling was done in 0.039" increments, beginning at the surface and continuing until a depth of 0.59". Figure 3-14 shows the mill machine set-up for sampling the bulk diffusion samples.

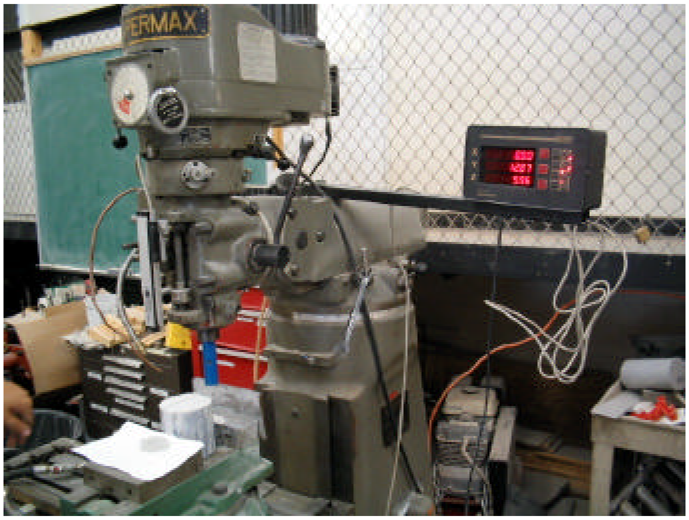


Figure 3-14. Sampling set-up for bulk diffusion samples

Once the samples were obtained from the bulk diffusion specimens, the chloride content was determined using the same RCT procedure used for the ponding test samples. Not all sample depths were tested. Instead, there were 8 depths tested, as required by ASTM C1556-03 (ASTM, 2003). For mix designs 7, 8 and 9, these depths were 0.039", 0.079”, 0.118”, 0.197”, 0.276”, 0.354”, 0.472” and 0.591”. Mix 10 was tested at different depths, ranging from 0.079” to 0.787”.

3.8 Road Salting Survey

Many components of this study were linked to specific regions of North Carolina. These divisions were made in an attempt to create information that will assist designers and maintenance officials who operate in geographically specific areas. In this study, the state was divided as shown in Figure 3-15.

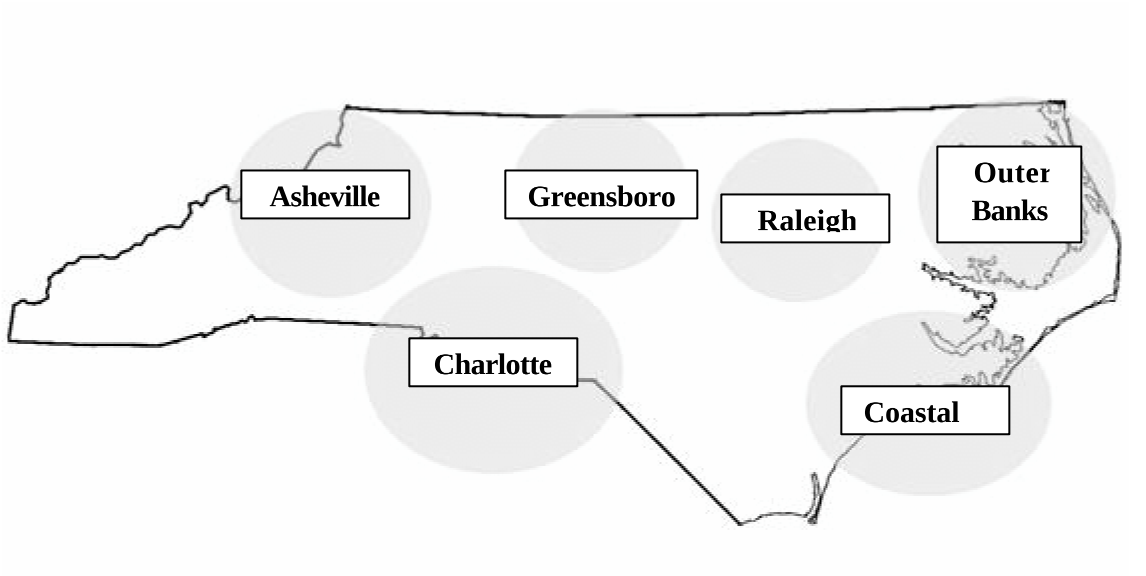


Figure 3-15. Geographic divisions used in road salting survey

Each region was studied in two ways. First, a survey of maintenance engineers operating in the region was made to determine the magnitude of road salt application in their districts. Secondly, bridges in each region were tested for the salt content of their structural concrete elements.

Because of specific climate and road conditions in the different regions of the state, it is known that the application rates and frequency of application can differ from the rates stated in the policy and Table 3-3. For this reason, a survey of highway maintenance personnel was performed to determine a more accurate and region specific application rate.

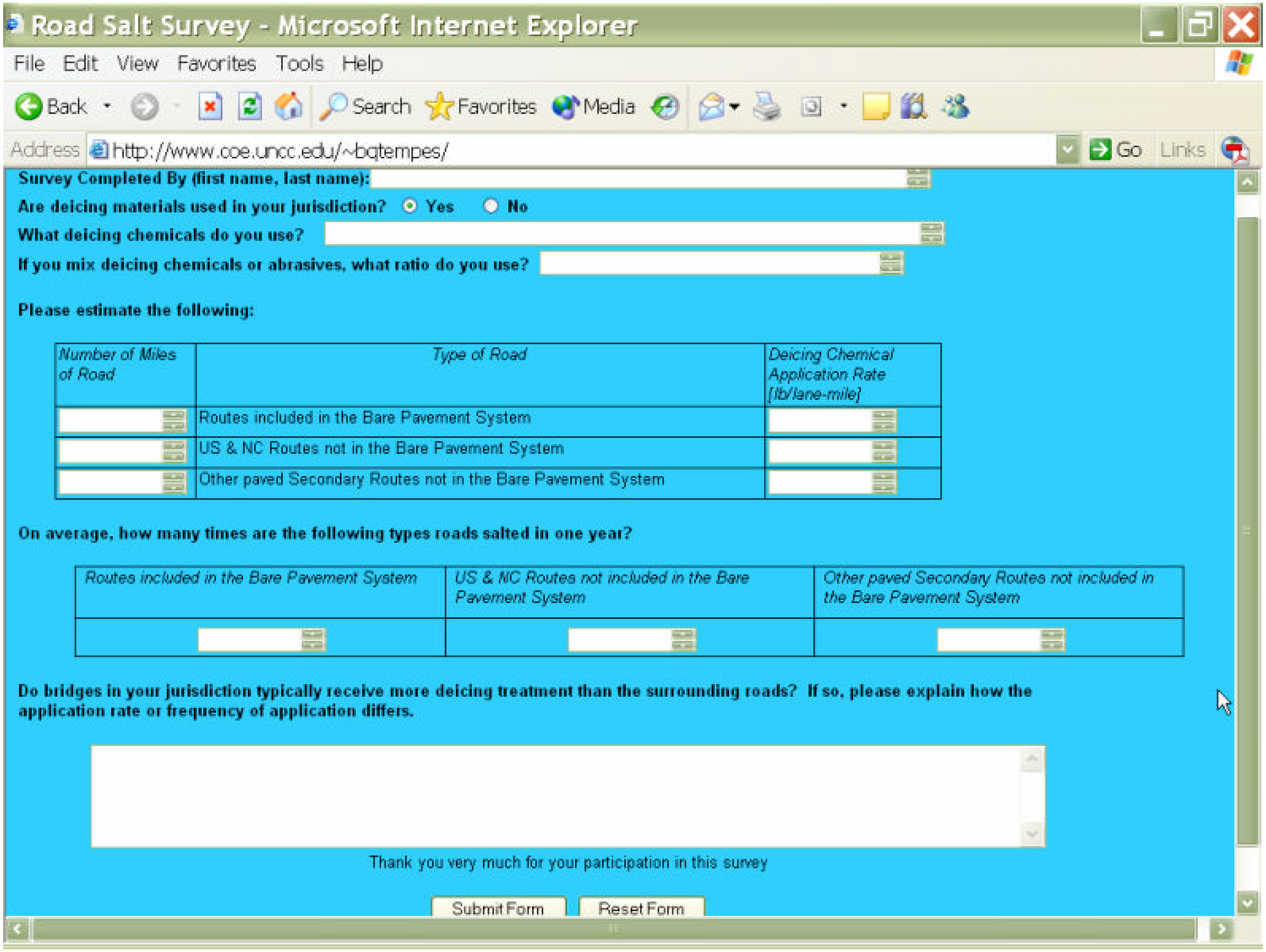
Table 3-3. Application rate for salt on Bare Pavement System roads

Temp.	*Pavement Condition*	*Type of Precipitation*	*Per Two-Lane Mile of Bare Pavement System*	*Instructions*
30°F. (-1°C.) & Above	Wet	Snow	300 lbs. of Salt	Do not plow as long as the salt is working. When the slush begins to stiffen, plow and re-apply at 200# salt as required.
30°F. (-1°C.) & Above	Wet	Sleet or Freezing Rain	200 lbs. of Salt	Re-apply salt at 200# as required.
25° to 30° F. (-4° to -1°C.)	Wet	Snow or Sleet	500 lbs. of Salt	Do not plow until slush begins to stiffen. After plowing, re-apply salt at 200#/two-lane mile.
25° to 30° F. (-4° to -1°C.)	Wet	Freezing Rain	300 lbs. of Salt	If subsequent applications are required, re-apply at 200# per two-lane mile.
20° to 25° F. (-7° to -4°C.)	Wet	Snow or Sleet	500 lbs. of Salt	Plow only when the slush begins to stiffen. Re-apply salt as required at a rate of 250# per two-lane mile.
15° to 20° F. (-9° to -7°C.)	Dry	Dry Snow	PLOW ONLY!!!	Do not apply chemicals. Treat hazardous areas with 1200 lbs. of 20:1 sand/salt or calcium chloride.
15° to 20° F. (-9° to -7°C.)	Wet	Wet Snow or Sleet	500 lbs. of 3:1 salt/calcium chloride mixture or salt moistened with calcium chloride	Do not plow until slush begins to stiffen. If roads become packed, treat hazardous areas with 1200# of 20:1 sand/salt or calcium chloride.
Below 15°F. (-9°C.)	Dry	Dry Snow or Sleet	PLOW ONLY!!!	Do not apply chemicals. Treat hazardous areas with 1200# of 20:1 sand/salt or calcium chloride.

From the questions in the survey, a yearly salt application rate for each region was determined. Because records of road salting are not maintained at the county, division or state level, this application rate is fairly subjective and often based on the best guess or memory of the person responding to the survey. The survey was administered electronically over the Internet. An email, sent to the county maintenance engineers by an NCDOT official, included an internet link to the survey form. Results were automatically collected by the UNCC research group.

The survey included questions on the following topics: type and mixture of deicing chemicals; miles of the Bare Pavement System serviced; miles of US & NC Routes, and other Secondary Routes serviced; salt application rate and frequency for Bare Pavement System, for US&NC routes, and for other Secondary routes; and information on whether bridges typically receive more deicing treatment than the surrounding roads, or not. A screen shot of the survey is shown in Figure 3-16.

The road salt survey was intended to be emailed to county maintenance engineers in all North Carolina counties west of Interstate 95. However, some responses were received from county maintenance engineers operating on the coast, which indicates that the survey was emailed to a larger target group than was intended.



http://www.coe.uncc.edu/~bqtempes/

Survey Completed By (first name, last name):

Are deicing materials used in your jurisdiction? ● **Yes** ○ **No**

What deicing chemicals do you use?

If you mix deicing chemicals or abrasives, what ratio do you use?

Please estimate the following:

Number of Miles of Road	*Type of Road*	*Deicing Chemical Application Rate [lb/lane-mile]*
	Routes included in the Bare Pavement System	
	US & NC Routes not in the Bare Pavement System	
	Other paved Secondary Routes not in the Bare Pavement System	

On average, how many times are the following types roads salted in one year?

Routes included in the Bare Pavement System	*US & NC Routes not included in the Bare Pavement System*	*Other paved Secondary Routes not included in the Bare Pavement System*

Do bridges in your jurisdiction typically receive more deicing treatment than the surrounding roads? If so, please explain how the application rate or frequency of application differs.

Thank you very much for your participation in this survey

Figure 3-16. Screen shot of the road salting survey

3.9 Bridge Sampling

The main purpose of this phase of the research was to assist in the verification and improvement of a chloride initiated bridge deterioration model. The basis of the model was described earlier. In summary, the model uses Fick's second law of diffusion to predict the ingress of chloride into concrete. In order to use the model, the surface chloride build-up coefficient, $\boldsymbol{k}$, or the constant surface concentration, C_0, and the diffusion coefficient, D, must be quantified experimentally or rationally. A field sampling program was devised for this project to collect data about these variables. The data was used to verify the model used to design the Virginia Dare Bridge as well as to develop criteria for modeling chloride ingress in bridges away from the coast.

In order to analyze the models, concrete powder samples from 28 bridges were collected. These bridges were distributed across the six geographic regions of North Carolina previously listed. Samples were typically removed from the deck and pier caps on inland bridges, and from the decks, columns and footings of coastal bridges. These locations were usually the site of maximum chloride contamination.

The selection of the 28 bridges was based on the following criteria:

- <u>Geographic location</u> – bridges were sampled near six urban centers in North Carolina: *Asheville*, in the Mountain region; *Charlotte*, in the Piedmont region; *Greensboro and Winston-Salem* in the Triad region; *Raleigh and Durham* in the Triangle region; *Wilmington* in the Coastal region; *Manteo* in the Outer Banks region.
- <u>Bridge component type</u> – bridge decks, pier caps, abutments, footings, and columns were considered.
- <u>Deck rating</u> – only bridges with a 4 or 5 deck rating were considered for field sampling. The exception was the Virginia Dare (Manteo Bypass) bridge, which has a very new deck. This limit on the condition of the bridge decks restricted the group to testing bridges that have been in service for long enough to have become contaminated with chloride, but not ones that will need replacement very soon.
- <u>Access</u> – the structural elements of interest had to be safely reachable by the research team. This restriction limited bridge decks on bridges with very heavy traffic or pier caps on bridges with very tall piers. In addition, there was a lower limit on the depth

of the girders that would allow space for the rotary hammers and other sampling equipment to work.

- The wearing surface – bridge decks were required to be of Portland cement concrete. Asphalt overlays were avoided.

Prior to sampling, each bridge was visited by NCDOT and UNCC research personnel to assess its suitability for sampling. Each sample collected had a unique collection datasheet, that contained information about the date and persons who collected the sample, a bridge identification number, the sample hole number and depths sampled, the salinity of the surrounding water if applicable, a small sketch about the hole's location and field notes. The specific structures sampled are listed in Table 3-4. The exact hole locations on the bridges from which samples were taken are provided in Tempest (2004).

The process for selecting a sampling site was different for each type of element. The general rules for each type of element are described below:

- Decks – the samples were taken along the length of a single span. One sample was taken four feet from each end of the span and one in the middle of the span. The holes were drilled between 16" and 24" off the curb. This location is in the gutter area, which was expected to have higher chloride concentration than the rest of the deck. Powder was usually not collected from inside the lane, where the surface would be affected by vehicular wearing.
- Pier Caps – samples were taken from the top surface of the pier cap. The location with the most visible drainage from the deck was selected for sampling. An attempt was made to take all samples at least five inches from any edge, in order to avoid the effect of boundary conditions.
- Footings – samples were taken from the top and the side of the footings. On the side, samples were taken as close to the high-tide mark as was practicable. On the top, samples were taken 16" from the edge, in the same vertical plane as the samples from the side.
- Columns – samples were taken from locations on the face that seemed to receive the most sea-water spray. In the case of the Bonner Bridge in Manteo, NC, samples were taken adjacent to the site of samples taken in 1986.

Table 3-4. A summary of bridges sampled

Area	Bridge Number	Bridge	Footing	Column	Deck	Pier Cap	Abutmen
Coastal	90013	Ocean Isle Bridge	6	1			
	90071	Holden Bridge	4				
	640013	Memorial Bridge	4				
	640027	US 421 Bridge	5		2		
	640011	US 117 Bridge	3		2		
Greensboro	400109	NC 6 Bridge			3	2	
	400221	Washington Street Bridge			3	1	
	400003	Elm Street Bridge			3		
	330395	US 311 Bridge			3	2	
	330275	Robinhood Road Bridge			3		
Raleigh	910494	Blue Ridge Road Bridge			3	2	
	910527	Edward's Mill Road Bridge			3		2
	310206	Cornwallis Road Bridge			2	2	
	310202	Alexander Drive over 147 Bridge			3	2	
	310100	Alexander Drive over I-40 Bridge			3		
	310247	Alexander Dr over Southern RR			3		2
Charlotte	590395	Hillside Street Bridge			3	2	
	590335	Hamilton Street Bridge			3	2	
	590164	Kings/Kennelworth Street Bridge			3		
	590317	Davidson Street Bridge			3	2	
	590138	Tryon Street Bridge			3	2	
Asheville	100324	Elk Mountain Road Bridge			3	2	
	100705	Haywood Road Bridge			3		
	100194	NC 191 Bridge			3	2	
	100295	Monte Vista Road Bridge			3	2	
	100783	Bennett Road Bridge			3		2
Outer Banks	270011	Bonner Bridge	3	3	6		
	270054	Virginia Dare	3	3	6		

Concrete powder was collected from structural components of the bridges listed above. Each hole was drilled for powder collection at discrete depth increments. Typically, the depths sampled were 1”, 2”, 3”, 4” and 5”. On some coastal footings, 7” and 9” samples were taken as well. Sample collection usually followed this procedure:

- Steps 1 and 2: The concrete surface was cleaned with a wire brush. The concrete surface was sprayed with alcohol and dried with paper towel as shown in Figure 3-17.
- Step 3: The sampling hole was predrilled with a 1¼" concrete bit in a rotary hammer, see Figure 3-18.
- Steps 4 and 5: the pre-drilled hole was brushed and then vacuumed (Figure 3-19).
- Step 6: Concrete powder from the desired depth was generated by drilling with a clean ¾" bit using a second rotary hammer. The larger sampling hole permitted the ¾" bit to avoid scraping the sides of the pre-drilled hole, see Figure 3-20.
- Step 7: The concrete powder was collected using a dustpan and a scoop that fit inside the small hole. The powder was stored in pre-labeled zip lock bags (Figure 3-20).
- Step 8: Finally, the holes were patched using a quick drying non-shrink mortar.

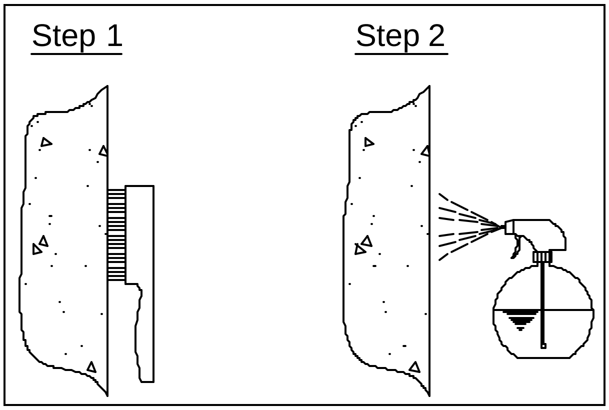


Figure 3-17. Cleaning the concrete surface with wire brush and alcohol

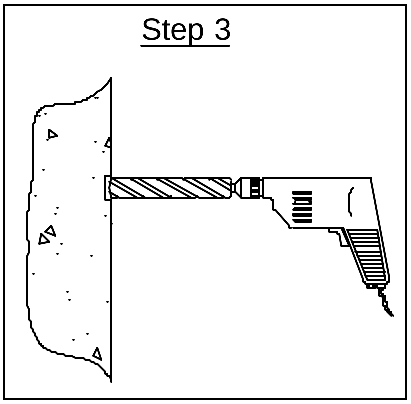


Figure 3-18. Pre-drilling the hole with a 1¼" concrete bit

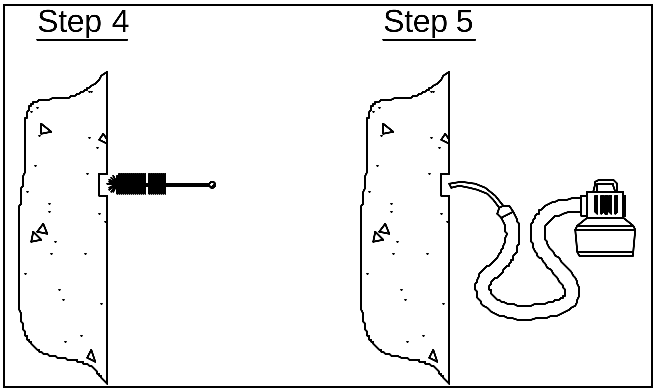


Figure 3-19. Cleaning and vacuuming the pre-drilled hole

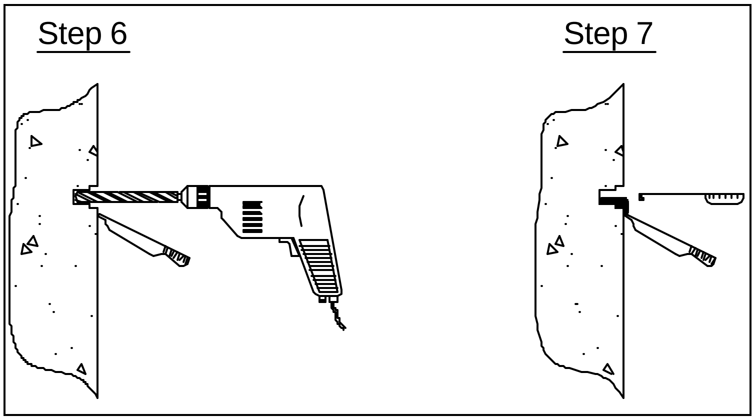


Figure 3-20. Drilling and collecting the concrete powder sample

Figure 3-21 presents a detailed view of the pre-drilling and drilling steps. Figures 3-17 through 3-20 illustrate the sampling technique for a vertical concrete surface, but the same steps were used for horizontal surfaces, such as bridge decks and the top of pier/pile caps. The required sample depth was reached by repeating Steps 2 through 7 as many times as needed. Between these steps, the drill bit and the collection tools were cleaned using alcohol, and dried using paper towels – to avoid powder sample contamination from dirty tools. The site was left in the condition shown in Figure 3-22. Once the samples were returned to the laboratory at UNCC, they were tested with either the RCT, or both the AASHTO T260 titration and the RCT methods, as described previously.

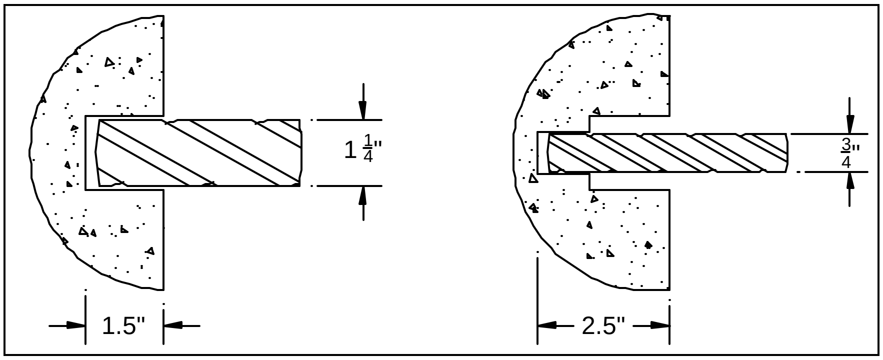


Figure 3-21. Detailed view of Steps 3 and 6

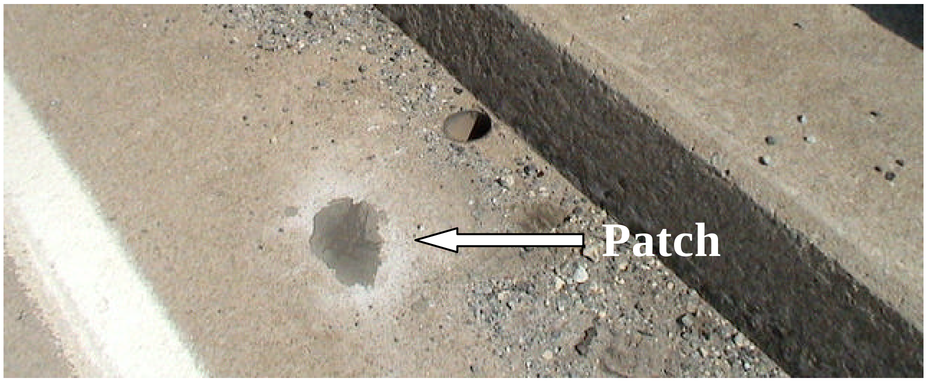


Figure 3-22. Final condition of sampled area

4. CHLORIDE DIFFUSION COEFFICIENTS

4.1 RCPT Results

All of the cylinders that were tested with the RCPT were exposed to identical curing conditions as the ponded slabs were: the cylinders were kept in a wet curing room for 14 days, and then moved to the environmental chamber, where the humidity was 50%, until they were tested. There were four mixes that were tested for both the RCPT and the chloride content of the ponded slabs after six months. These were mixes 2, 3, 7 and 10, the only mix designs that did not have supplementary cementitious materials that would increase the density of the concrete. The remaining mix designs, as well as the four that were tested at six months, were tested after one year of ponding.

The results from the six month and one year RCPT tests for mix design 7D were selected to be presented. The results for all the RCPT's can be seen in Bledsoe (2005). All of the final coulomb values were corrected based on Equation 4-1, which corrects the coulombs passed for a cylinder greater than 3.75" in diameter (ASTM, 2002).

$$Q_s = Q_x * (\frac{3.75}{x})^2 \tag{4-1}$$

where:

Q_s = Charge passed (coulombs) through a 3.75" diameter cylinder.

Q_x = Charge passed through a cylinder with a diameter of x.

x = Diameter (inches) of cylinder tested.

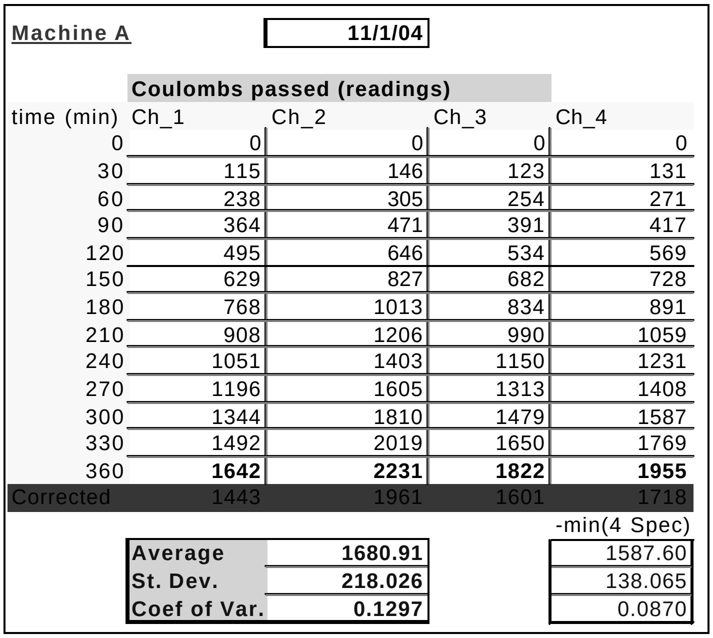
As mentioned previously, Hooton et al. (2001) suggested shortening the RCPT to 30 minutes to eliminate the effect of heat build-up in the cells, and then multiplying the coulombs passed in 30 minutes by 12 to simulate the entire six hour test. Based on this recommendation, all RCPT tests were performed for 6 hours, but the results were corrected using the 30-minute readings.

In order to analyze the RCPT data, a simple spreadsheet program was developed. Table 4-1 shows the results obtained from the six month RCPT of mix 7D prior to correction. As it can be seen, the average coulomb count passed for all four specimens was 1,681, with a coefficient of variation (COV) of 12.97%. After the outlier (1,961 coulombs) was removed, the average became 1,588 coulombs, with a coefficient of variation of 8.70%; clearly satisfying the standard required maximum COV of 12.5%.

Table 4-2 shows the corrected data (also given in Figure 4-1), resulting in average coulombs passed of 1,297 and a coefficient of variation of 6.5% (Hooton et al., 2001). Based on these results, mix 7D was classified as having low permeability (ASTM, 2002).

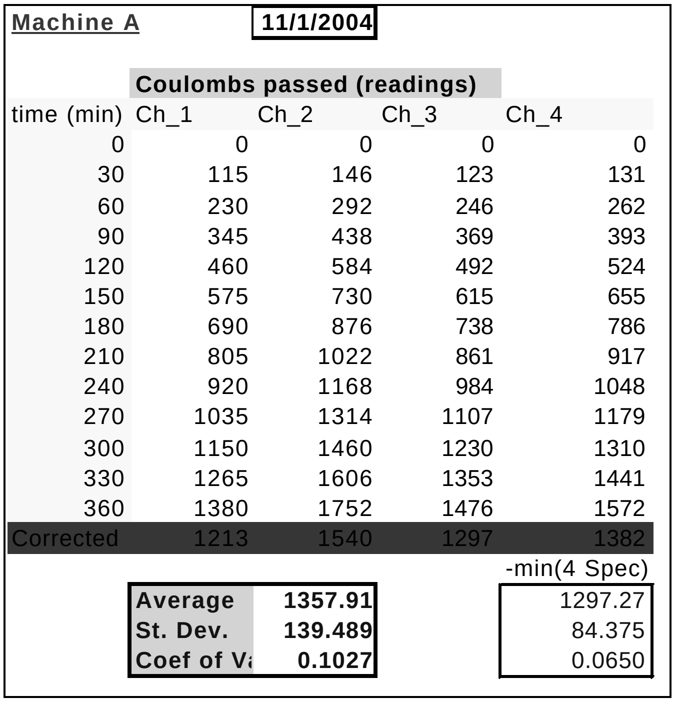
Table 4-3 shows the corrected results from the one year test for mix 7D. This test yielded somewhat lower results than the six month test, which is to be expected because of the longer curing time. The RCPT results for all of the final mix designs can be seen in Table 4-4. It is important to notice that the other six-month result were lower than the same mix design's one-year RCPT results, for which no clear explanation can be provided at this time.

Table 4-1. RCPT results for mix 7D six-month test



Machine A		**11/1/04**		
	Coulombs passed (readings)			
time (min)	Ch_1	Ch_2	Ch_3	Ch_4
0	0	0	0	0
30	115	146	123	131
60	238	305	254	271
90	364	471	391	417
120	495	646	534	569
150	629	827	682	728
180	768	1013	834	891
210	908	1206	990	1059
240	1051	1403	1150	1231
270	1196	1605	1313	1408
300	1344	1810	1479	1587
330	1492	2019	1650	1769
360	**1642**	**2231**	**1822**	**1955**
Corrected	1443	1961	1601	1718
				-min(4 Spec)
	Average	**1680.91**		1587.60
	St. Dev.	**218.026**		138.065
	Coef of Var.	**0.1297**		0.0870

Table 4-2. RCPT results for mix 7D six-month test, corrected values



Machine A **11/1/2004**

	Coulombs passed (readings)			
time (min)	Ch_1	Ch_2	Ch_3	Ch_4
0	0	0	0	0
30	115	146	123	131
60	230	292	246	262
90	345	438	369	393
120	460	584	492	524
150	575	730	615	655
180	690	876	738	786
210	805	1022	861	917
240	920	1168	984	1048
270	1035	1314	1107	1179
300	1150	1460	1230	1310
330	1265	1606	1353	1441
360	1380	1752	1476	1572
Corrected	1213	1540	1297	1382

		-min(4 Spec)
Average	**1357.91**	1297.27
St. Dev.	**139.489**	84.375
Coef of V	**0.1027**	0.0650

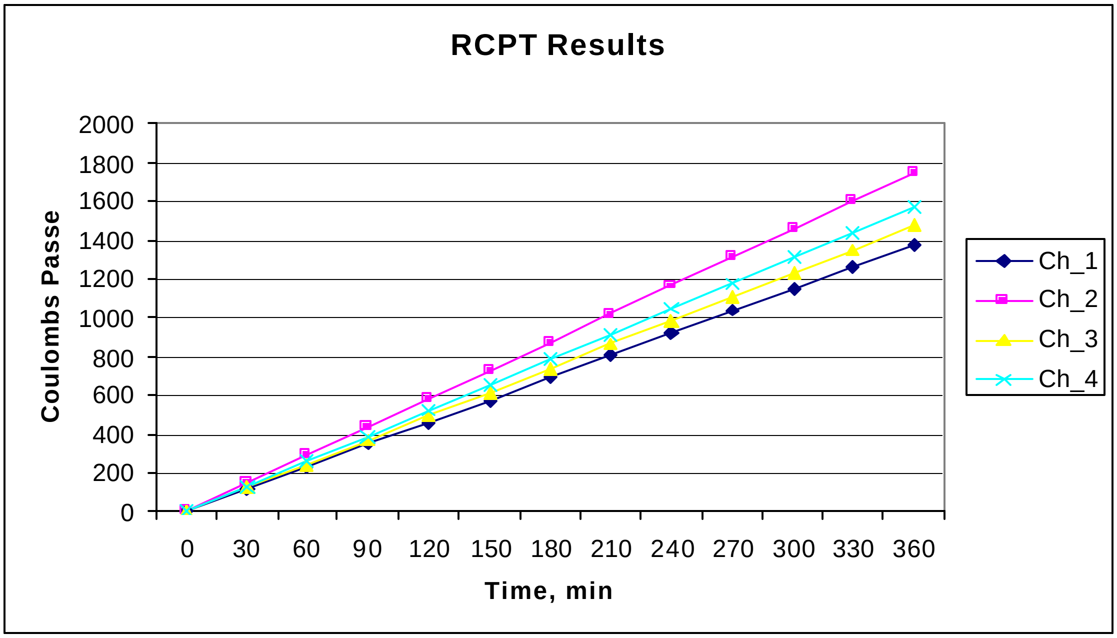


Figure 4-1. Mix 7D six-month RCPT test results, corrected values

Table 4-3. RCPT results for mix 7D one-year test, corrected values

Machine A		**3/2/05**		
	Coulombs passed (readings)			
time (min)	Ch_1	Ch_2	Ch_3	Ch_4
0	0	0	0	0
30	115	116	138	130
60	230	232	276	260
90	345	348	414	390
120	460	464	552	520
150	575	580	690	650
180	690	696	828	780
210	805	812	966	910
240	920	928	1104	1040
270	1035	1044	1242	1170
300	1150	1160	1380	1300
330	1265	1276	1518	1430
360	1380	1392	1656	1560
Corrected	1213	1223	1455	1371
				-min(4 Spec)
	Average	**1315.72**		1269.14
	St. Dev.	**117.878**		88.451
	Coef of V	**0.0896**		0.0697

Table 4-4. RCPT results for final mix designs, corrected values

Mix Number	**Coulombs Passed**
2f (6 mth)	2732
3e (6 mth)	3632
7d (6mth)	1297
10d (6mth)	3231
1f	4092
2f	3635
3e	4598
4d	2415
5e	3632
6e	2770
7d	1269
8c	4127
9a	5084
10d	3892

4.2 Ponding Test Results

At the beginning of this project it was determined that the ponding tests would have to last longer than the AASHTO specified 90 days. Based on recommendations from NCDOT, the test was lengthened to one year for all of 10 mix designs, with 4 of the mix designs being sampled after six months as well. The slabs were dried overnight and then sampled with a hammer drill. The holes were then filled with fast setting epoxy, and the slabs were re-ponded and stored in the curing chamber for the remaining six months.

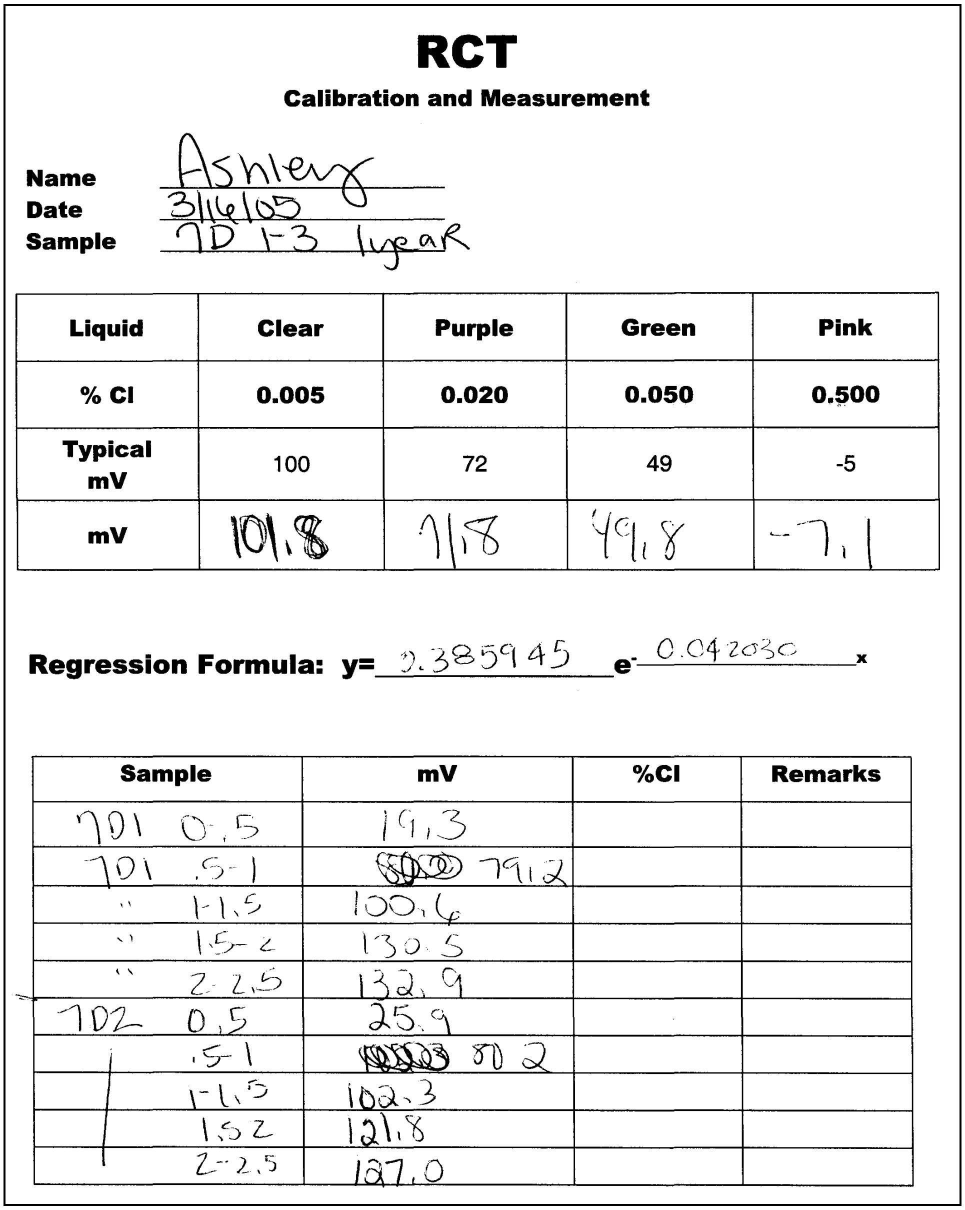
The ponded slabs were sampled at five depths: 0.625”-0.5”, 0.5”-1”, 1”-1.5”, 1.5”-2”, and 2”-2.5”. The AASHTO T277 test method only requires that samples be obtained from the first two depths listed above (AASHTO, 2002). However, the remaining depths were sampled as well, in order to estimate the diffusion coefficient for the slab. Figure 4-2 shows the drilling of the slabs and the collection of powder samples from each depth. The powder samples collected from the slabs were tested for chloride content with the Rapid Chloride Test kit.

Figure 4-2. Sampling of slabs with hammer drill

After all of the ponded slabs had been sampled and tested for chloride content, one or two slabs from each mix design, with questionable results, was re-sampled using the profile grinding technique described by Hooton et al. (1999). This was accomplished by placing the slab onto the milling machine and using a diamond-tipped core bit to sample the slab in 0.039” (1 mm) increments (Figure 4-3). Each of the slabs was sampled to a depth of 0.591”, with only eight depths being tested for chloride content.

Figure 4-3. Sampling of slabs by profile grinding

As was the case for the RCPT, the RCT had a data sheet to make sure that all the results were kept in hard copy as well as electronic copy. Figure 4-4 shows an example of one of these data sheets. The calibration readings for the probe are recorded, and the equation used to determine the chloride content based on Germann Instruments user manual. This equation is unique each time the probe is calibrated because it is dependant solely on the mV readings from the calibration liquids.



RCT

Calibration and Measurement

Name Ashley
Date 3/16/05
Sample 7D 1-3 1year

Liquid	**Clear**	**Purple**	**Green**	**Pink**
% Cl	**0.005**	**0.020**	**0.050**	**0.500**
Typical mV	100	72	49	-5
mV	101.8	71.8	49.8	-7.1

Regression Formula: y= 0.385945 **e-** 0.042030 **x**

Sample	**mV**	**%Cl**	**Remarks**
701 0.5	19.3		
701 .5-1	79.2		
" 1-1.5	100.6		
" 1.5-2	130.5		
" 2-2.5	132.9		
702 0.5	25.9		
.5-1	80.2		
1-1.5	102.3		
1.5 2	121.8		
2-2.5	127.0		

Figure 4-4. RCT data sheet

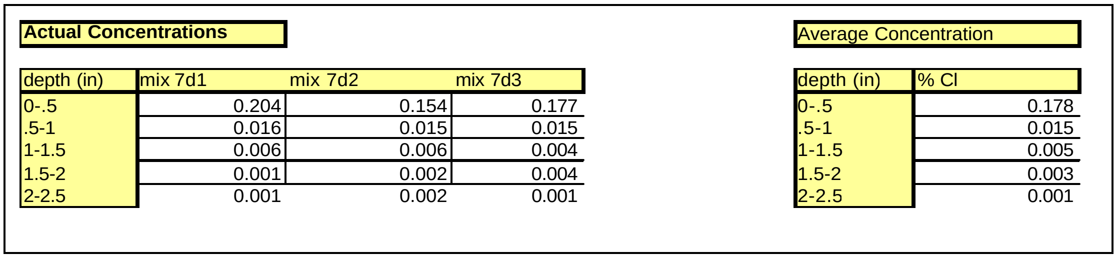
Once the mV readings were recorded for each sample, the results were entered into an Excel spreadsheet and the percent chloride was calculated using the regression formula seen in Figure 4-4 (e.g. for this sample the formula was $y = 0.385945e^{-0.042030}$). A graph was then drawn displaying the percent chloride versus depth for the slab. Once all three ponded slabs were sampled and tested, the average concentration was calculated and used to determine the diffusion coefficient and surface chloride content.

This was done using the ‘genfit’ function from Mathsoft’s MathCAD 11.0, Enterprise Edition. The ‘genfit’ function is capable of fitting an arbitrary equation to a set of data points (Tempest, 2004). The way the program was set up, all the known variables for Equation 2-2 were defined, with the chloride content listed as a percentage of concrete mass and the depth listed in millimeters. Reasonable guesses were then made for the diffusion coefficient and the surface chloride content. The function then determined the two unknowns using a minimization of the sum of the squares (Tempest, 2004). All the results from the ponding test can be found in (Bledsoe, 2005).

To illustrate the analysis of the ponding test data, the results from mix design 7D are presented and analyzed here. Mix 7D was sampled and tested both after six months of ponding, and again after one year of ponding. The one year test results will be presented here. Table 4-5 shows the chloride concentration at each depth for each of the three ponded slabs from mix 7D. These concentrations were calculated by subtracting the background chloride content, which was 0.001% for this mix, from the measured chloride content. The chloride contents from each slab were then averaged to determine the average chloride content for mix 7D at each depth. These values were then used to calculate the diffusion coefficient in MathCAD.

Once the results are entered into the Excel spreadsheet, a graph is drawn to show the chloride profile with depth. Figure 4-5 shows this profile for mix 7D. The graph shows that the results provided a typical chloride diffusion profile.

Table 4-5. Chloride concentrations for mix 7D



Actual Concentrations

depth (in)	mix 7d1	mix 7d2	mix 7d3
0-.5	0.204	0.154	0.177
.5-1	0.016	0.015	0.015
1-1.5	0.006	0.006	0.004
1.5-2	0.001	0.002	0.004
2-2.5	0.001	0.002	0.001

Average Concentration

depth (in)	% Cl
0-.5	0.178
.5-1	0.015
1-1.5	0.005
1.5-2	0.003
2-2.5	0.001

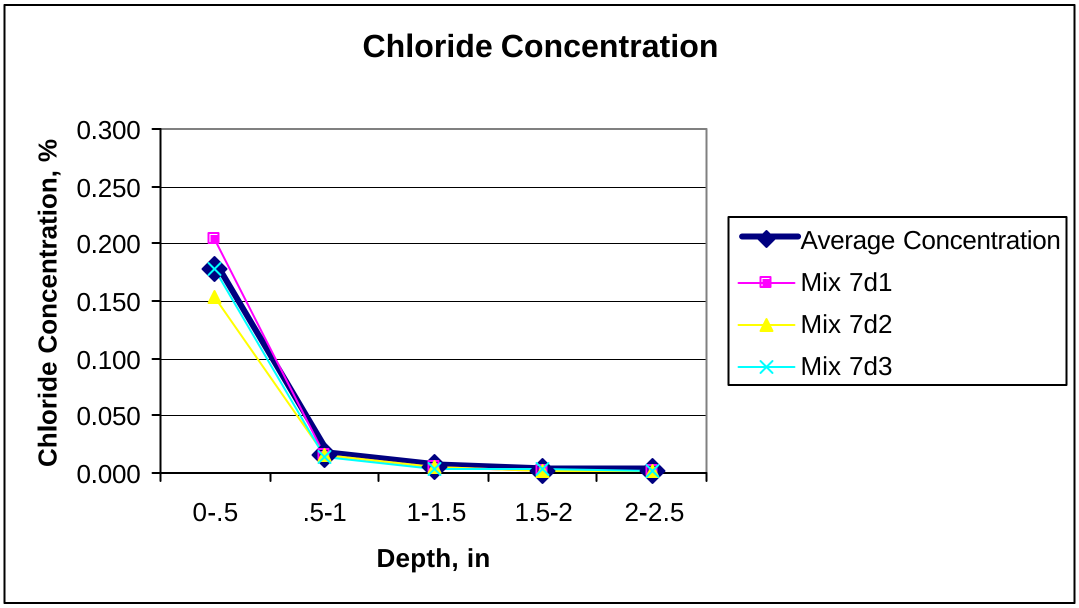


Figure 4-5. Chloride profile for mix 7D

There are three inputs required for the 'genfit' function in MathCAD. The first is an array relating the chloride concentration, in percent, to the depth of the sample, in millimeters. The second is an array of guesses to start the curve fitting process, and the third is an array of three equations. These three equations include the solution to Fick's Second Law, Equation 2.2, the partial derivative of this equation with respect to the surface concentration parameter, and the partial derivative of Equation 2.2 with respect to the diffusion coefficient. The two partial derivatives are provided in Equations 4-2 and 4-3.

$$\frac{\partial}{\partial C_0} = \left[1 - erf\left[\frac{1}{4}x\frac{4^{\frac{1}{2}}}{(D\cdot t)^{\frac{1}{2}}}\right]\right] \tag{4-2}$$

$$\frac{\partial}{\partial D} = \frac{1}{4}\cdot\frac{C_0}{p^{\frac{1}{2}}}\cdot\exp\left(\frac{1}{4}\cdot\frac{x^2}{u_i\cdot t}\right)\cdot x\cdot\frac{4^{\frac{1}{2}}}{(D\cdot t)^{\frac{1}{2}}}\cdot t \tag{4-3}$$

where:

C=concentration of a chemical species [kg/m^3]

x=a linear distance [meters]

t=time [seconds]

D =diffusion coefficient [m^2/second]

C_0 =the surface concentration of a chemical species

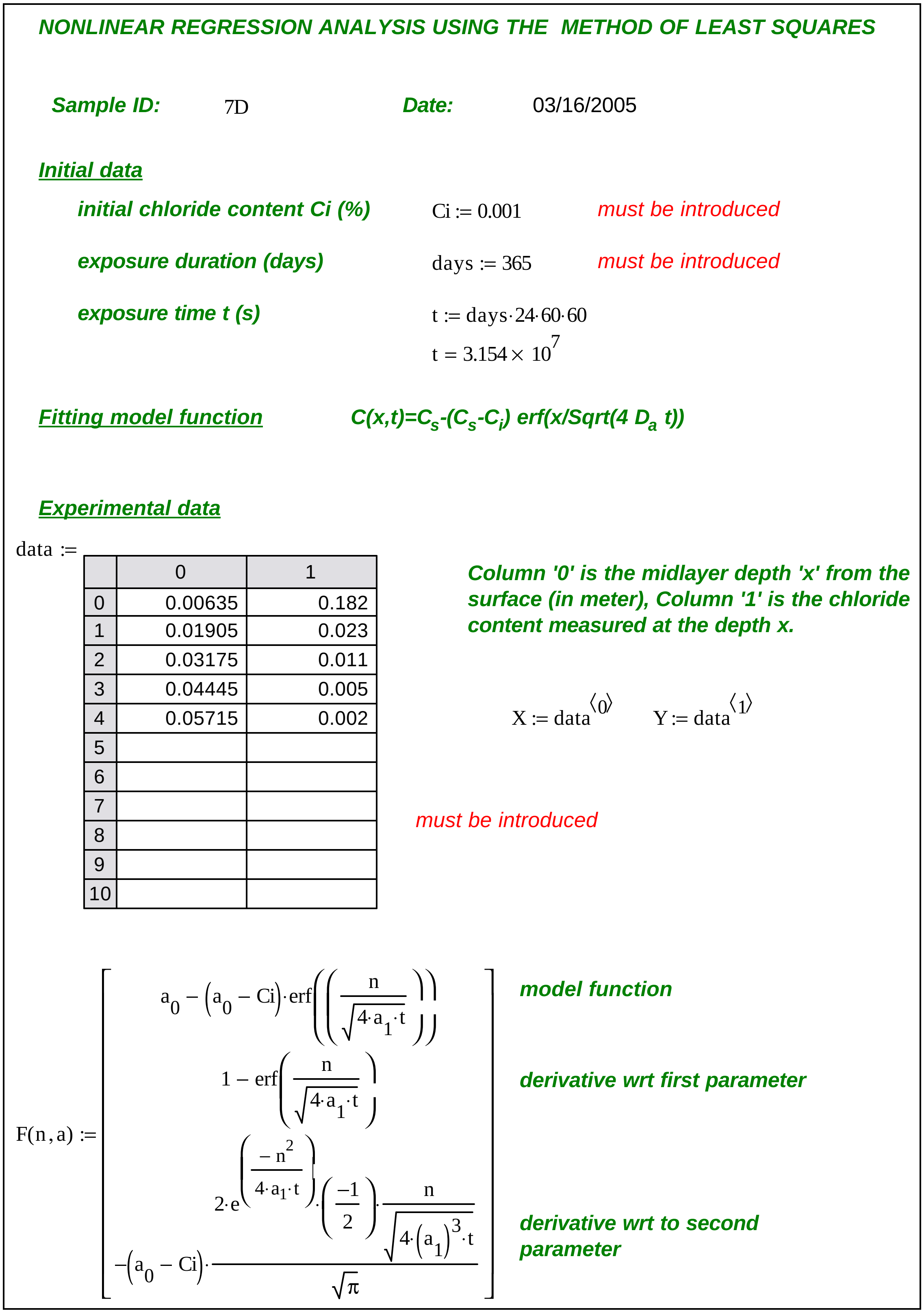
erf =the error function

Once the MathCAD file was created, it was used for each mix design to determine the surface concentration and diffusion coefficient. The chloride concentrations at each depth were entered into the program as an array, as seen in Figure 4-6. The depths are listed on the left-hand side in meters. The depth used is the midpoint of the actual sampling depth. For example, the first depth is 0.25". This is the midpoint between 0" and 0.5". The depths then increase by 0.5" for each subsequent depth.

The other input values required are the background chloride content from the control slab, the time t that the slab had been ponded, and the array of guesses for the surface chloride content and the diffusion coefficient. The guesses for these two parameters were 1% and 0.097 in^2/year (2x10^{-12} m^2/sec), respectively. Once these values were input into the program, the actual values of the surface chloride content and the diffusion coefficient were calculated. For mix 7D these values were 0.333% and 0.085 in2/year (1.74x10^{-12} m^2/sec), respectively. A screenshot of the MathCAD program file for mix 7D, in its entirety, can be seen in Figures 4-6 and 4-7. The original MathCAD program file is available upon request.

Table 4-6 shows the diffusion coefficients for all of the final mix designs. As it can be seen, mix designs 1, 4, 5, 6, 7 and 9 yielded very low diffusion coefficients. These results will be further analyzed later. The effect of concrete age reduces the diffusion coefficient, and this is proven by the 4 mixes that were tested at 6 months versus 1 year.

The profile grinding technique yielded improved correlation between the slabs within four of the ten final mix designs: mixes 2f, 3e, 4d, and 10d. For these four mix designs, the diffusion coefficient was determined by averaging the two slabs that were sampled normally, and the third one that was sampled via profile grinding.



NONLINEAR REGRESSION ANALYSIS USING THE METHOD OF LEAST SQUARES

Sample ID: 7D **Date:** 03/16/2005

Initial data

initial chloride content Ci (%) $Ci := 0.001$ *must be introduced*

exposure duration (days) $days := 365$ *must be introduced*

exposure time t (s) $t := days\cdot 24\cdot 60\cdot 60$

$t = 3.154\times 10^{7}$

Fitting model function **C(x,t)=C$_s$-(C$_s$-C$_i$) erf(x/Sqrt(4 D$_a$ t))**

Experimental data

data :=

	0	1
0	0.00635	0.182
1	0.01905	0.023
2	0.03175	0.011
3	0.04445	0.005
4	0.05715	0.002
5		
6		
7		
8		
9		
10		

Column '0' is the midlayer depth 'x' from the surface (in meter), Column '1' is the chloride content measured at the depth x.

$X := data^{\langle 0\rangle}$ $Y := data^{\langle 1\rangle}$

must be introduced

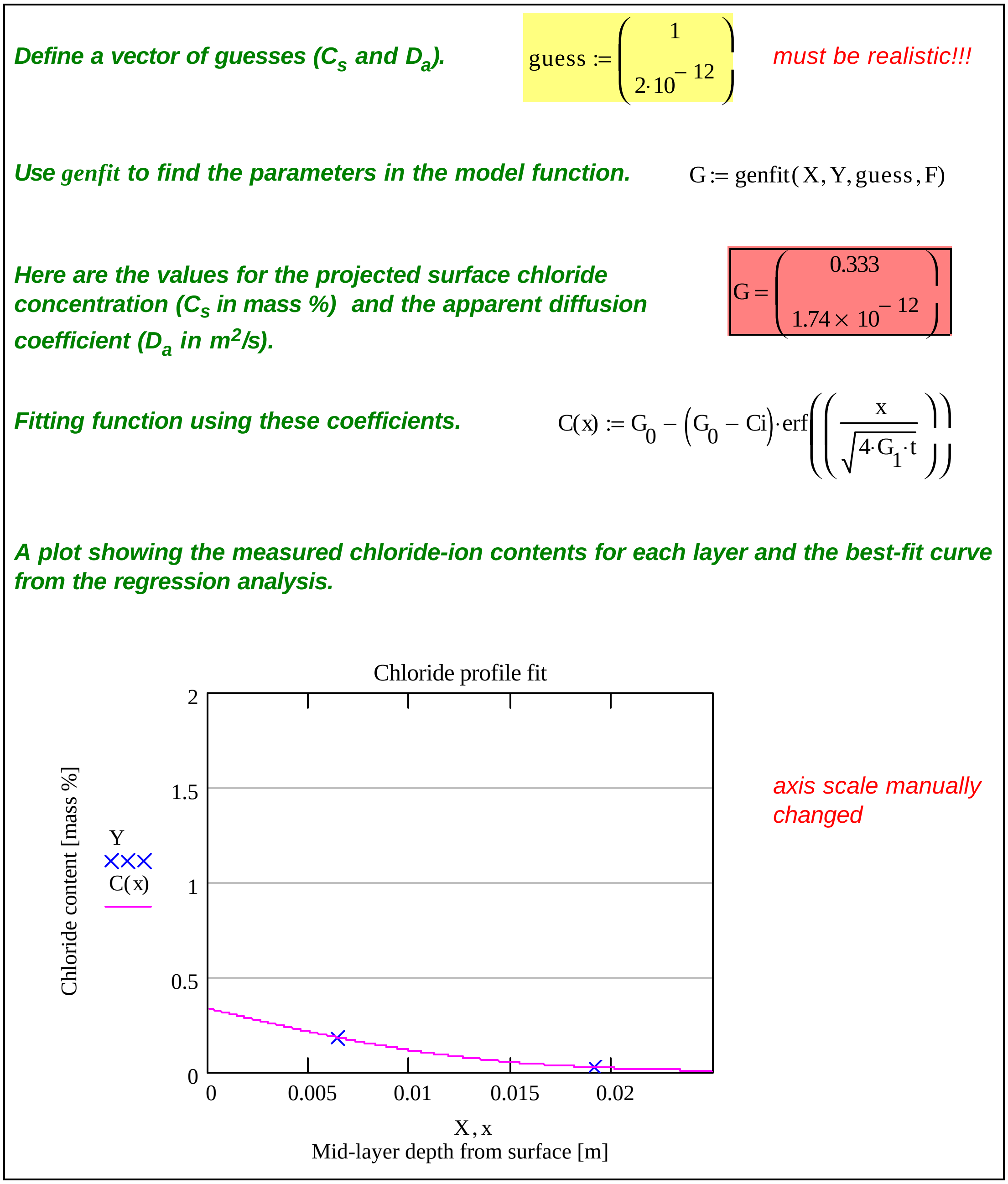
$$F(n,a) := \begin{bmatrix} a_0 - (a_0 - Ci)\cdot \mathrm{erf}\left(\left(\frac{n}{\sqrt{4\cdot a_1\cdot t}}\right)\right) \\ 1 - \mathrm{erf}\left(\frac{n}{\sqrt{4\cdot a_1\cdot t}}\right) \\ -(a_0 - Ci)\cdot\dfrac{2\cdot e^{\left(\frac{-n^2}{4\cdot a_1\cdot t}\right)}\cdot\left(\frac{-1}{2}\right)\cdot\frac{n}{\sqrt{4\cdot(a_1)^3\cdot t}}}{\sqrt{\pi}} \end{bmatrix}$$

model function

derivative wrt first parameter

derivative wrt to second parameter

Figure 4-6. Part 1 of MathCAD file



Define a vector of guesses (C_s and D_a).

$$\text{guess} := \begin{pmatrix} 1 \\ 2 \cdot 10^{-12} \end{pmatrix}$$

must be realistic!!!

Use genfit to find the parameters in the model function.

$$G := \text{genfit}(X, Y, \text{guess}, F)$$

Here are the values for the projected surface chloride concentration (C_s in mass %) and the apparent diffusion coefficient (D_a in m^2/s).

$$G = \begin{pmatrix} 0.333 \\ 1.74 \times 10^{-12} \end{pmatrix}$$

Fitting function using these coefficients.

$$C(x) := G_0 - (G_0 - Ci) \cdot \text{erf}\left(\left(\frac{x}{\sqrt{4 \cdot G_1 \cdot t}}\right)\right)$$

A plot showing the measured chloride-ion contents for each layer and the best-fit curve from the regression analysis.

axis scale manually changed

Figure 4-7. Part 2 of the MathCAD file (1.0×10^{-12} m^2/sec = 0.0487 in^2/year)

Table 4-6. Diffusion coefficients for final mix designs

Mix Number	Diffusion Coefficient in^2/year (*10^{-12}, m^2/sec)
1f	0.058 (1.20)
2f (6 months)	0.276 (5.67)
2f (1 year)	0.227 (4.66)
3e (6 months)	0.451 (9.26)
3e (1 year)	0.355 (7.29)
4d	0.055 (1.13)
5e	0.051 (1.05)
6e	0.099 (2.04)
7d (6 months)	0.105 (2.16)
7d (1 year)	0.095 (1.96)
8c	0.151 (3.11)
9a	0.087 (1.78)
10d (6 months)	0.278 (5.71)
10d (1 year)	0.180 (3.70)

4.3 RMT Results

Rapid migration tests were performed on four mix designs. The cylinders from two of these mix designs, mixes 7 and 8, were not poured at the same time as the test specimens for the ponding test and RCPT. The final mix results were almost identical to the mix results from the original test batch to within a percent for the air content, and a half of an inch for the slump. The test cylinders for mixes 9 and 10 were poured at the same time as the other test specimens, so the cylinders contain identical materials. The only difference was that the cylinders were kept in the wet curing room until they were tested. This amounted to approximately 11 months in wet cure, whereas mixes 7 and 8 only spent about 6 months in wet cure. It is not known how this affected the results, although it is expected that curing time has an effect on the results.

A data acquisition system, DAQ, was used during the rapid migration test to record the temperature of the catholyte solution, the applied voltage, and the current through the specimen. These readings were recorded in a text file which was then inserted into Excel. Once the test was completed, the test specimen was split with a chisel and the split face was sprayed with silver nitrate. The depth of chloride penetration was then determined by measuring the white precipitate that formed. All of this information was then used to calculate the non-steady-state diffusion coefficient using Equation 3-4.

Also calculated was the rate of penetration. To calculate this, the average chloride penetration depth was divided by the applied voltage multiplied by the time of the test in hours. This gives a number with the units of in/V-hr (mm/V-hr), and is used to correlate the RMT data to the RCPT and ponding test (Hooton et al., 2001). A minimum of three specimens were tested from each of the four mix designs, as per the Nordtest standard. The migration coefficient and rate of penetration was calculated for each specimen, and then averaged together to determine the mix design's coefficient and penetration rate (Table 4-7). The full results from each migration test are provided in Bledsoe (2005).

Table 4-7. Results from the RMT tests

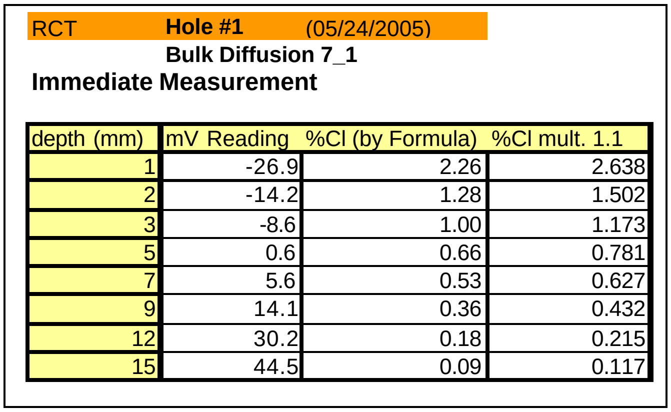
Mix Number	D_{nssm} in^2/yr (*10^{-12}, m^2/sec)	Rate of Penetration *10^{-4}, in/V-hr (mm/V-hr)
7	0.244 (5.02)	6.69 (0.017)
8	0.303 (6.22)	7.09 (0.018)
9	0.114 (2.35)	3.54 (0.009)
10	0.367 (7.55)	9.84 (0.025)

4.4 Bulk Diffusion Results

Bulk diffusion tests were performed on the same four mix designs as the RMT. From each of these designs, two samples were immersed in the NaCl solution as required by ASTM C1556-03 (ASTM, 2003). The samples from the four mixes were tested for their chloride content using the RCT test equipment, and then the diffusion coefficient for each specimen was calculated using the same MathCAD file used for the ponding test. The diffusion coefficient for each mix was then determined by averaging the two diffusion coefficients per mix.

The results from mix 7 will be presented and analyzed here, while the rest of the results are provided in Bledsoe (2005). The two specimens for mix 7 were kept in the solution for a total of 103 days. The time is important because it is one of the variables in the equation for calculating the diffusion coefficient. Once the specimens were removed from the solution, they were sampled within 24 hours. Every millimeter up to 15 mm was sampled, but not all of them were tested for chloride content. The depths that were tested for specimen 7_1 were 0.039", 0.079", 0.118", 0.197", 0.276", 0.354", 0.472", and 0.591". For specimen 7_2 the depths were the same except 0.394" and 0.433"were tested instead of 0.472" and 0.591" because the core bit broke in the cylinder, preventing further sample collection. The effect of this on the results for the diffusion coefficient calculations was minimal because the chloride profile was already well developed within the first 0.433". The RCT results for specimen 7_1 can be seen in Table 4-8 with the corresponding graph in Figure 4-8.

Table 4-8. RCT results for bulk diffusion specimen 7_1 (1 mm = 0.039")



RCT **Hole #1** (05/24/2005)
Bulk Diffusion 7_1
Immediate Measurement

depth (mm)	mV Reading	%Cl (by Formula)	%Cl mult. 1.1
1	-26.9	2.26	2.638
2	-14.2	1.28	1.502
3	-8.6	1.00	1.173
5	0.6	0.66	0.781
7	5.6	0.53	0.627
9	14.1	0.36	0.432
12	30.2	0.18	0.215
15	44.5	0.09	0.117

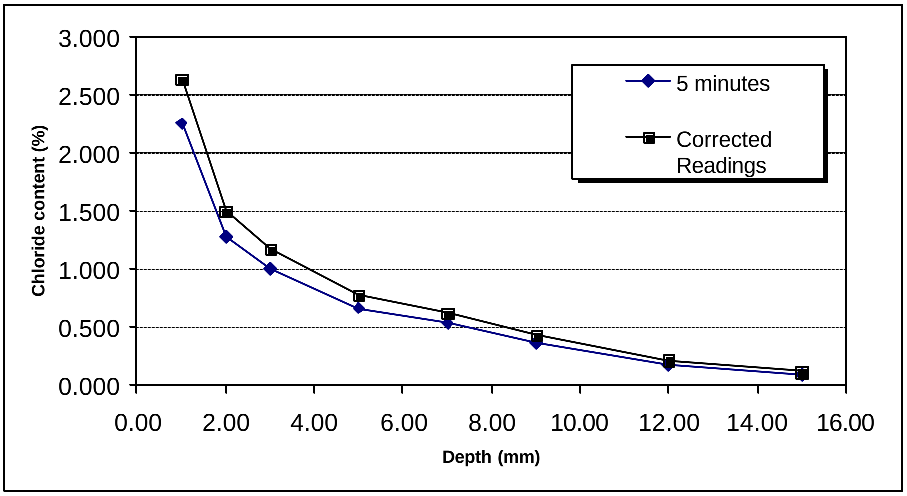


Figure 4-8. Graph for bulk diffusion results for specimen 7_1 (1 mm = 0.039")

These results were then entered into the same MathCAD program illustrated in Figures 4-6 and 4-7 to obtain the diffusion coefficient. The diffusion coefficient for this specimen was 0.068 in^2/yr (1.40x10^{-12} m^2/s). This value was averaged with the diffusion coefficient calculated for specimen 7_2 (0.075 in^2/yr, or 1.54x10^{-12} m^2/s) to determine mix 7's diffusion coefficient (0.072 in^2/yr, or 1.47x10^{-12} m^2/s). This was done for the three other mixes as well, and the results can be seen in Table 4-9.

Table 4-9. Diffusion coefficients from bulk diffusion tests

Mix Number	Diffusion Coefficient in^2/yr (*10^{-12}, m^2/sec)	Average Diff. Coeff. in^2/yr (*10^{-12}, m^2/sec)
7_1	0.068 (1.40)	0.072 (1.47)
7_2	0.075 (1.54)	
8_1	0.178 (3.65)	0.158 (3.24)
8_2	0.138 (2.83)	
9_1	0.411 (8.43)	0.373 (7.65)
9_2	0.335 (6.88)	
10_1	0.086 (1.77)	0.079 (1.62)
10_2	0.072 (1.47)	

4.5 RCPT Results for Trial Batches

After the RCPT had been completed for all of the final concrete mixes, the decision was made to test the intermediate batches of concrete from the mix designs. The intermediate batches were the result of trial and error, as the additives controlling the air content and slump were varied to obtain the final mix, which corresponded to the original NCDOT specifications. Two 4"x8" cylinders were made for each trial mix, and these were the cylinders that were tested. These additional RCPT tests were done to investigate if the air content or slump affects the RCPT values.

The same procedure was used for the tests in that the 30 minute coulomb reading was multiplied by 12 in order to obtain the total coulombs passed. The number of trial batches varied for each mix design, ranging from 3 to 5 trials. Mix 9 did not have any trial mixes because it was obtained directly from a bridge construction site. Table 4-10 provides the RCPT results for all intermediate and final batches.

Table 4-10. Comprehensive RCPT results

Mix Number	Coulombs Passed	Air Content [%]	Slump [in.]
1a	5600	10.0	3.0
1b	6138	3.0	1.25
1c	5244	5.5	3.0
1d	3966	2.0	0.25
1e	4177	6.3	3.0
1f	4092	6.5	2.5
2a	5414	7.0	4.5
2b	7165	7.0	7.5
2d	4802	5.8	1.0
2f	3635	7.0	2.0
3a	3428	4.0	6.25
3c	3185	3.0	1.0
3e	4598	7.5	4.0
4a	1635	9.5	2.5
4b	1768	2.5	2.5
4c	2257	5.0	8.0
4d	2415	5.5	6.0
5a	2303	4.0	3.5
5b	1382	3.0	4.25
5c	1477	4.0	4.5
5d	2591	7.0	7.5
5e	3632	8.0	8.0
6a	4530	4.5	2.25
6b	3709	4.5	5.5
6c	3227	5.5	6.0
6d	2415	5.5	2.5
6e	2770	5.0	2.5
7a	2334	6.0	8.75
7b	1427	4.5	12.0
7c	1208	4.0	7.0
7d	1269	4.5	7.5
8a	1322	6.0	8.75
8b	2358	5.0	6.0
8c	4127	6.0	7.0
9a	5084	4.5	2.5
10a	4106	5.3	2.0
10b	3403	6.0	2.5
10c	4785	5.0	1.5
10d	3892	8.5	2.5

Note: see Section 5.3 for more information on the effects of air content and slump

5. COMPARISON OF CHLORIDE DIFFUSION TEST RESULTS

5.1 Ponding and RCPT Comparison

The main purpose of this phase of the project was to determine if there was a correlation between the RCPT test results and the diffusion coefficients obtained from the AASHTO T259 salt ponding test.

There were several correlations that were attempted for the RCPT and Ponding test results. The first was relating the coulombs passed versus the calculated diffusion coefficient for each mix. When this was done for all of the mix designs, the correlation using a linear regression analysis was poor, with an r^2 value of 0.22. When the mixtures were separated based on whether or not they contained supplementary cementitious materials, the correlation improved somewhat. For the mix designs not containing supplementary cementitious materials, the linear correlation only increased to 0.30 (see Figure 5-1). A fourth order polynomial was used in the correlation for mix designs that contained the supplementary admixtures, and an r^2 value of 0.67 was obtained (see Figure 5-2). As disappointing as these results were, they were expected based on past research; therefore, after this initial comparison, further attempts were made to seek a better correlation.

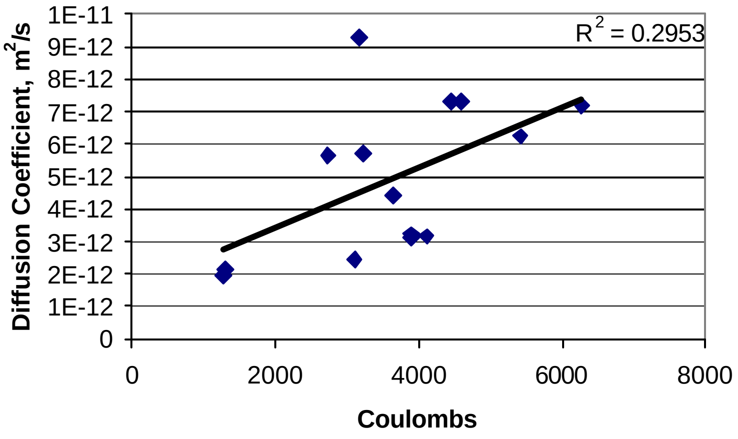


Figure 5-1. Correlation for mix designs without supplementary admixtures ($1.0\text{x}10^{-12}$ m^2/sec = 0.0487 in^2/year)

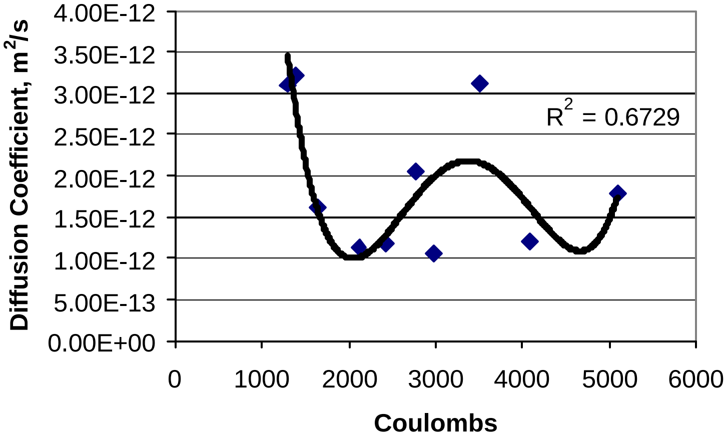


Figure 5-2. Correlation for mix designs with supplementary admixtures (1.0×10^{-12} m^2/sec = 0.0487 in^2/year)

In order to perform these additional correlations, several values had to be calculated. These values included the effective diffusion coefficient for the RCPT test from Equations 2-3 and 2-4; the total integral chloride content from the ponding test results as illustrated in Figure 2-1; and the depth to 0.1% chloride content from the ponding test, which was interpolated from the graphs resulting from the test results. These values, as well as the coulombs passed and ponding diffusion coefficients for all mixes are in Table 5-1. The mix designs with an asterisk beside them are the mixtures which contained supplementary cementitious materials.

There were numerous correlations attempted using the data from Table 5-1. The best correlations were found when the mixes were separated based on their cementitious materials content. To make these correlations, some data was disregarded when it was determined that the information was erroneous. The reasons for these few erroneous results vary, from possible contamination of samples, to mistakes made during testing. The results from the profile grinding of the slabs did not improve correlations significantly; however, there were some very interesting results obtained indicating that further research needs to be done to determine if profile grinding is a viable alternative to the traditional sampling techniques in use today.

Table 5-1. Values used for correlation between RCPT and Ponding tests
($1.0\text{x}10^{-12}$ m^2/sec = 0.0487 in^2/year)

Mix Number	Coulombs Passed	RCPT Diffusion Coefficient, m^2/s	Diffusion coefficient, m^2/s	Total Integral Cl Content (%)	Depth to 0.1% Cl Penetration, mm
2a(6 mth)	6275	1.63669E-11	7.19E-12	0.2365	17.9
2f(6 mth)	2732	7.93292E-12	5.67E-12	0.218	17.7
3a(6 mth)	4454	1.24678E-11	7.32E-12	0.3585	22.6
3e(6 mth)	3164	1.00768E-11	9.26E-12	0.3305	23.6
7d(6 mth)	1297	2.04355E-12	2.16E-12	0.1355	13.5
10a(6 mth)	3888	1.14166E-11	3.13E-12	0.3135	17.7
10d(6 mth)	3231	9.13407E-12	5.71E-12	0.185	16.3
1f*	4092	1.11399E-11	1.20E-12	0.152	14.3
2a	5414	1.40928E-11	6.23E-12	0.35	23.8
2f	3635	1.0085E-11	4.66E-12	0.2865	19.1
3a	3097	9.59938E-12	2.43E-12	0.219	16.3
3e	4598	1.22865E-11	7.29E-12	0.504	29.2
4a*	1635	2.43618E-12	1.61E-12	0.031	0
4d*	2415	3.27742E-12	1.13E-12	0.031	0
5a*	2113	3.16073E-12	1.13E-12	0.091	11.5
5b*	1382	2.14379E-12	3.22E-12	0.261	18.3
5e*	2985	1.00768E-11	1.05E-12	0.071	8.9
6e*	2770	8.02715E-12	2.04E-12	0.269	17.5
7d	1269	2.00979E-12	1.96E-12	0.1125	11.9
8a*	1301	2.07294E-12	3.11E-12	0.132	9.9
8c*	3502	4.92484E-12	3.11E-12	0.239	17.1
9*	5084	1.33666E-11	1.778E-12	0.193	15.8
10a	4106	1.1172E-11	3.158E-12	0.6125	26.2
10d	3892	1.06798E-11	3.70E-12	0.967	31.75

As it can be seen in Table 5-1, mixes 2, 3, 7, and 10 did not contain supplementary cementitious materials (SCM). Therefore, all the results from these tests were compiled and correlations were attempted. When the data was analyzed, several data points were seen to be outliers. For instance, the total integral chloride and depth to 0.1% chloride concentration for mix designs 10A and 10D for the one year tests are unusually high compared to the other mixes. This was possibly due to the fact that the ponding solutions on these slabs regularly had higher salinity readings than the other slabs for no apparent reason. The depth to 0.1% chloride content for mix design 3E was also considerably higher than the other mixes, while the coulombs passed for the six month test on 2A were much higher than the others.

When these values were taken out of the correlation spreadsheet, the accuracy of the correlations increased dramatically. The three correlations that were used for these mix designs were the coulombs passed versus the depth to 0.1% chloride, coulombs passed versus total integral chloride content, and the diffusion coefficient from the RCPT results versus the total integral chloride content. The results from these correlations and their respective r^2 values can be seen in Figures 5-3 through 5-5.

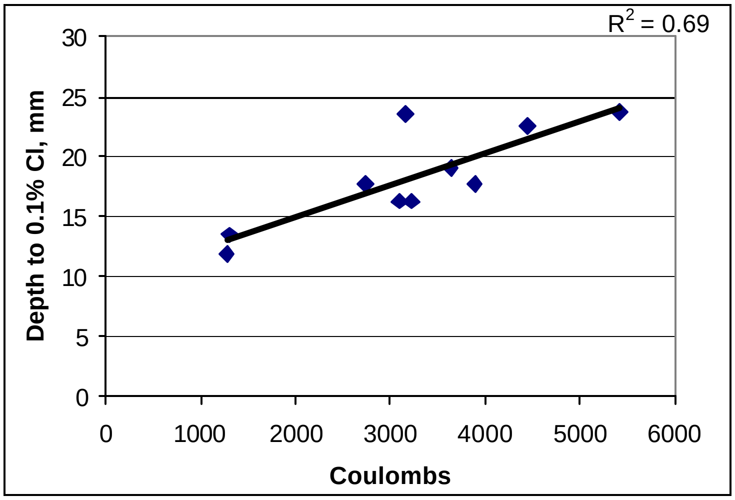


Figure 5-3. Coulombs passed vs. depth to 0.1% chloride, no SCM (1 mm=0.039")

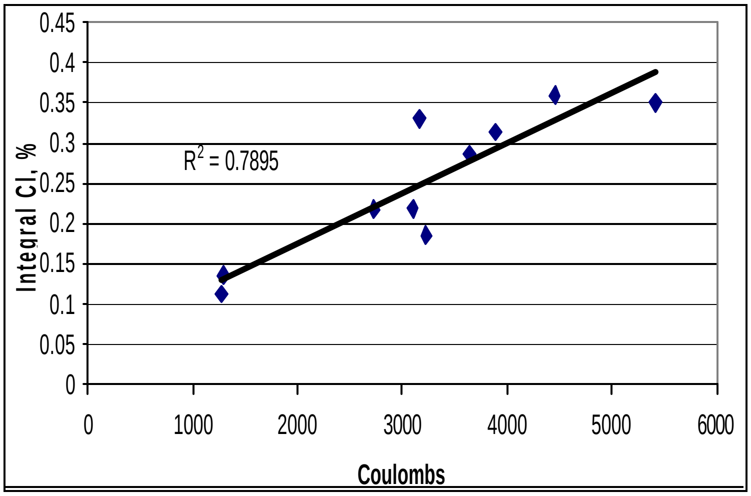


Figure 5-4. Coulombs passed vs. integral chloride, no SCM

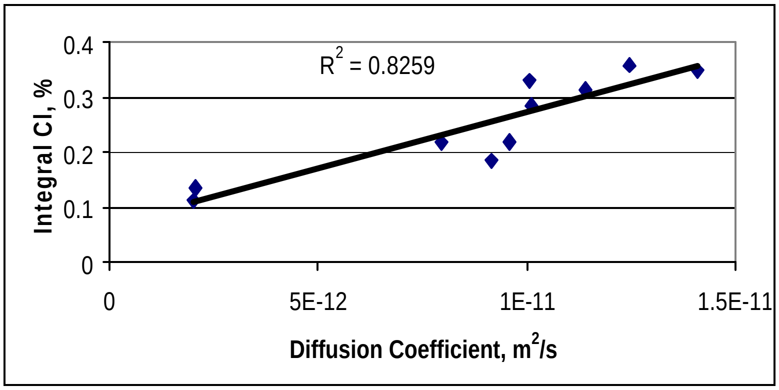


Figure 5-5. RCPT diffusion coefficient vs. integral chloride, no SCM
($1.0x10^{-12}$ m^2/sec = 0.0487 in^2/year)

The best correlation that was obtained was with the effective diffusion coefficient calculated from the RCPT results and the total integral chloride content from the ponding test, with an r^2 of 0.83. The correlation between the coulombs passed and the total integral chloride was also high, with an r^2 of 0.79.

The correlations for the mix designs that contained supplementary cementitious materials were not as good as those just presented. As with the mixtures without supplementary admixtures, there were a few outliers that were disregarded. There were only two mix designs that were excluded for the correlations: mix 4A and 4D. As it can be seen in Table 5-1, these two mix designs had very little chloride penetration, and the total integral chloride content was negligible. In fact, the chloride concentration never reached 0.1%, making the data unusable for correlation purposes.

Even with these two mixes removed from the database, the correlations were not that good. The best correlation was between the RCPT effective diffusion coefficient and the Ponding diffusion coefficient with an r^2 of 0.64. The next best correlation was between the coulombs passed during the RCPT test and the diffusion coefficient calculated from the ponding test, with an r^2 of 0.63. Figures 5-6 and 5-7 show these two correlations, both of which were done with a fourth order polynomial!

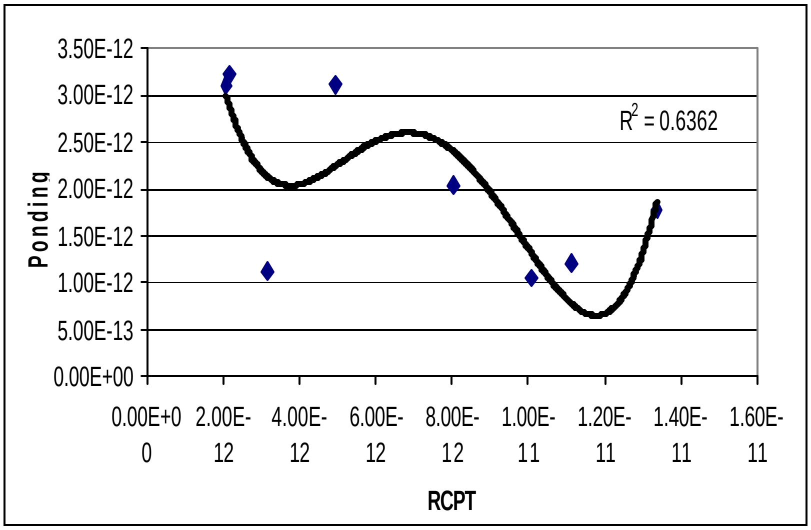


Figure 5-6. RCPT vs. Ponding diffusion coefficient, with SCM
($1.0\text{x}10^{-12}$ m^2/sec = 0.0487 in^2/year)

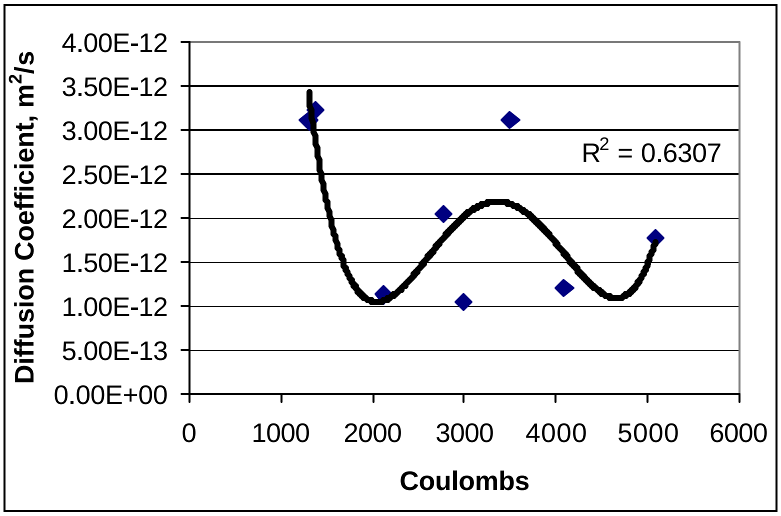


Figure 5-7. Coulombs passed vs. diffusion coefficient, with SCM

($1.0x10^{-12}$ m^2/sec = 0.0487 in^2/year)

With the poor correlation exhibited by the concrete mixes with supplementary admixtures, a decision was made to separate the mixes even further. Therefore, the mix designs with silica fume and DCI-S corrosion inhibitor (SFCI), which happened to be used together in the mixes, were separated and further correlations were attempted. The mix designs that contained these two admixtures were 4, 5 and 8. Once again, mixes 4A and 4D were disregarded due to the extremely low chloride content measured from the ponding test. Therefore, mix 5 and 8 provided five data points to attempt a correlation, which does not really provide a large enough statistical database.

The correlations that were obtained from the coulombs passed versus the ponding diffusion coefficient are extremely high. Note that this correlation is a second order polynomial (see Figure 5-8). Well, the same conclusion can not be made from the RCPT and ponding diffusion coefficients (see Figure 5-9).

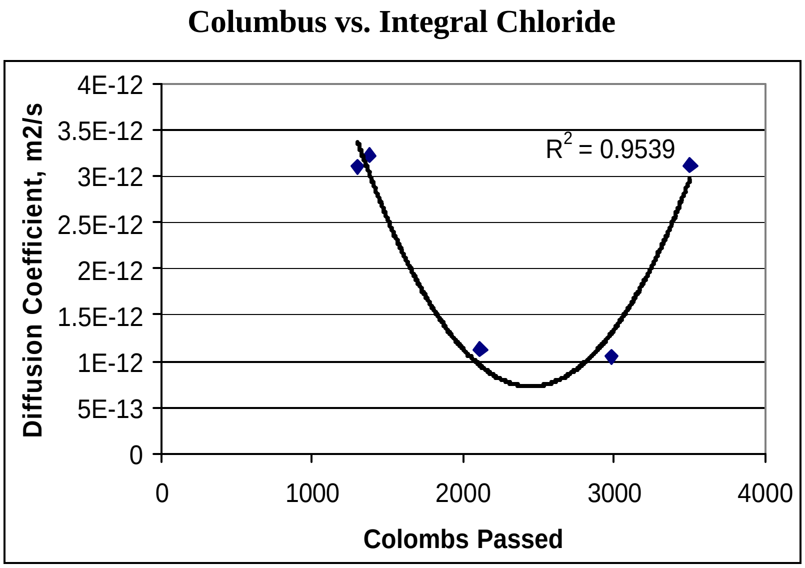


Figure 5-8. Coulombs passed vs. diffusion coefficient, with SFCI ($1.0x10^{-12}$ m^2/sec = 0.0487 in^2/year)

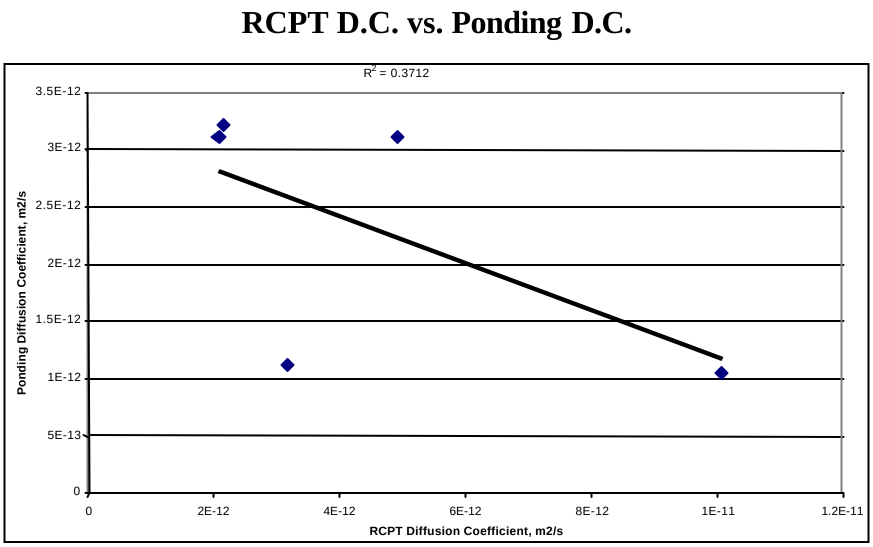


Figure 5-9. RCPT vs. Ponding diffusion coefficients, with SFCI ($1.0x10^{-12}$ m^2/sec = 0.0487 in^2/year)

5.2 Bulk Diffusion and Rapid Test Comparison

The bulk diffusion and rapid migration test were performed on four mix designs. An attempt was made to compare the data obtained from these tests, to determine if there was a better correlation than was found for the ponding versus RCPT tests. The results for the four mix designs can be seen in Table 5-2. The ponding and RCPT results were also included for reference.

Table 5-2. Results for four mix designs used for bulk diffusion and RMT tests ($1.0\text{x}10^{-12}$ m^2/sec = 0.0487 in^2/year)

Mix Number	RCPT Coulombs Passed	RCPT D.C.	Ponding D.C.	Bulk Diffusion D.C.	RMT M.C.	RMT Rate of Penetration
7	1269	2.01E-12	1.96E-12	1.47E-12	5.02E-12	0.017
8	3502	4.92E-12	3.11E-12	3.24E-12	6.22E-12	0.018
9	5084	1.34E-11	1.78E-12	8.43E-12	2.35E-12	0.009
10	3892	1.07E-11	3.70E-12	1.62E-11	7.55E-12	0.025

Once these results were obtained, correlations were attempted between the bulk diffusion results and the RCPT and RMT results. The correlations that were attempted were between the bulk diffusion coefficient and the RMT rate of penetration and the RCPT coulombs passed. The first attempt yielded a weak correlation for the former, giving an r^2 of 0.22, but yielded a decent correlation for the later, giving an r^2 of 0.63. For each of the correlations, there was one outlier that dramatically affected the results. Mix 9 was this outlier. The bulk diffusion sample for mix 9 was exposed to the solution for 90 days and then removed and placed into a sealed plastic bag because it could not be sampled right away. The sampling of the specimen took place 10 months later. It is possible that diffusion continued throughout this time, giving erroneous results for the bulk diffusion test. For this reason, the bulk diffusion results for mix 9 were disregarded from further correlation leaving only 3 data points. When this was done, the correlations improved, with the r^2 values increasing to 0.99 and 0.69, respectively. Figures 5-10 and 5-11 show the correlations that were obtained.

The last correlation that was attempted was between the ponding test and the RMT results. This correlation is important because some researchers believe that the RMT could possibly replace the RCPT as the rapid test of choice. The correlation was attempted between the ponding diffusion coefficient and the rate of penetration from the RMT test. Mix 9 was not disregarded from this correlation, since the bulk diffusion results were not used. The correlation that was obtained yielded an r^2 of 0.64 for an exponential regression curve (see Figure 5-12), which could be improved with a larger data set available for different mix designs.

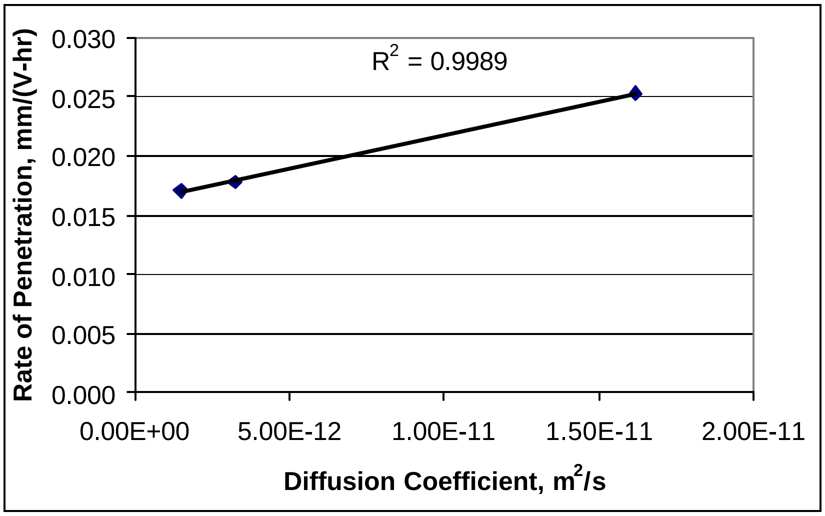


Figure 5-10. Bulk diffusion vs. RMT (1.0×10^{-12} m^2/sec = 0.0487 in^2/year)

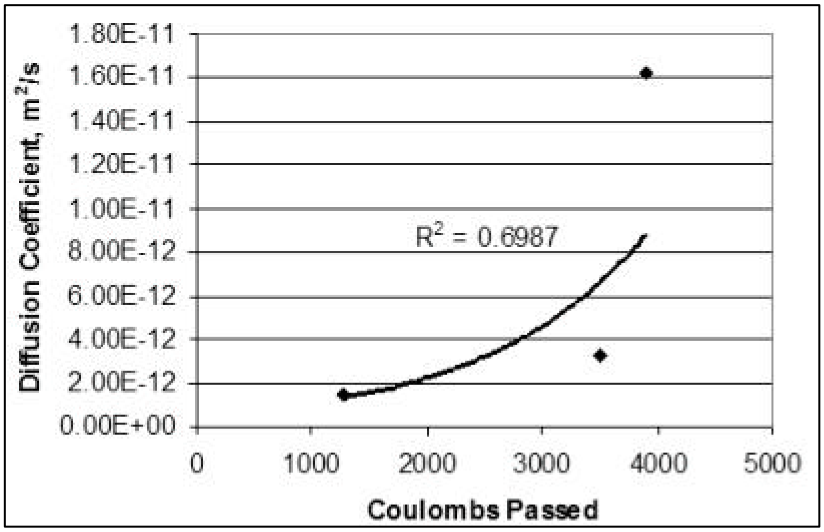


Figure 5-11. Bulk diffusion vs. RCPT (1.0×10^{-12} m^2/sec = 0.0487 in^2/year)

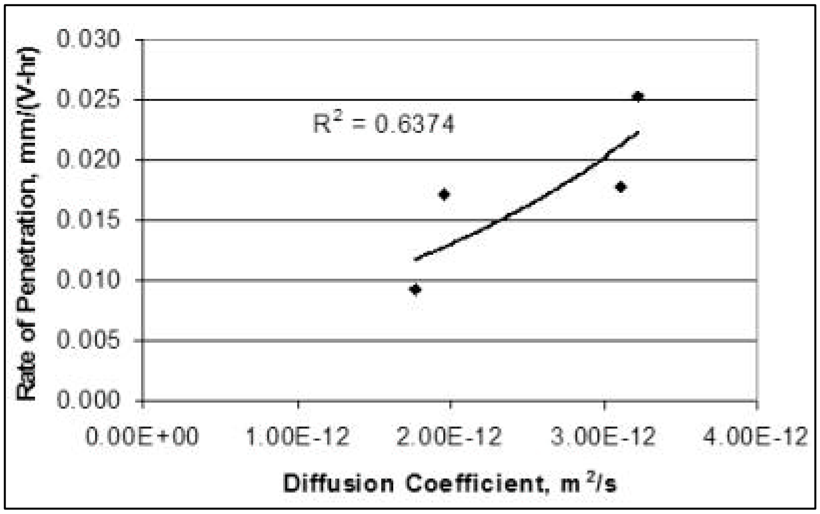


Figure 5-12. Ponding vs. RMT ($1.0\text{x}10^{-12}$ m^2/sec = 0.0487 in^2/year)

5.3 Effect of Air Content and Slump on RCPT Values

When concrete mix designs are specified for various construction projects, the air content and slump are always an important part of this specification. There is not, however, a set limit above or below the specified values at which the concrete is not allowed to be poured into the forms. With the trial mix cylinders that were left over, an attempt was made to determine if the air content or slump had an effect on the RCPT values for a specific mix design.

After all of the mix designs were analyzed based on slump and air content, each mix design was analyzed separately. This yielded some very interesting results. Almost every mix design experienced good correlations between either the slump and coulombs passed, or the air content and coulombs passed. There were a few, however, that yielded good correlations for both the air content and slump versus the coulombs passed. Mix designs 4, 5 and 7 were these mixes. As it can be seen in Figures 5-13 through 5-15, the r^2 values for air content versus coulombs passed, and the slump versus coulombs passed were high for these three mix designs. This leads the researchers at UNCC to believe that the slump and air content does have a significant effect on the permeability of concrete mixes, but further research is needed to validate and quantify this finding.

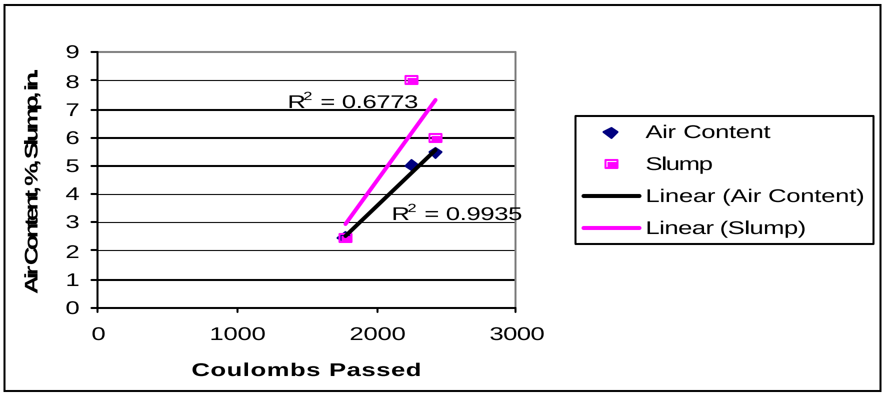


Figure 5-13. Coulombs versus air content and slump for Mix 4

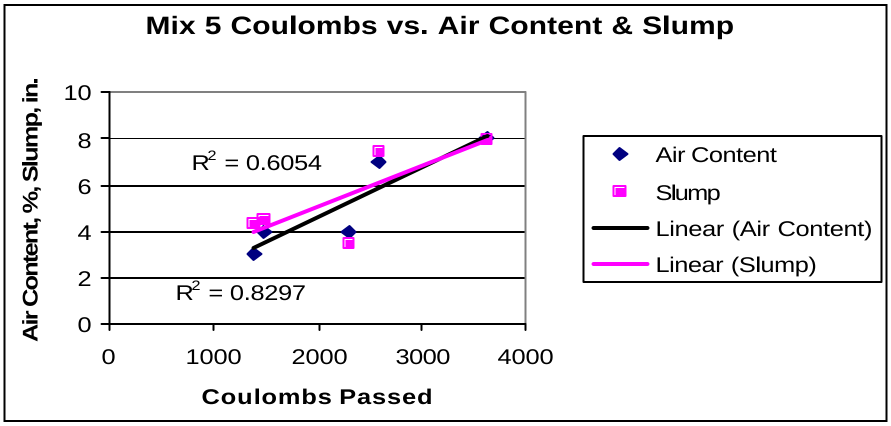


Figure 5-14. Coulombs versus air content and slump for Mix 5

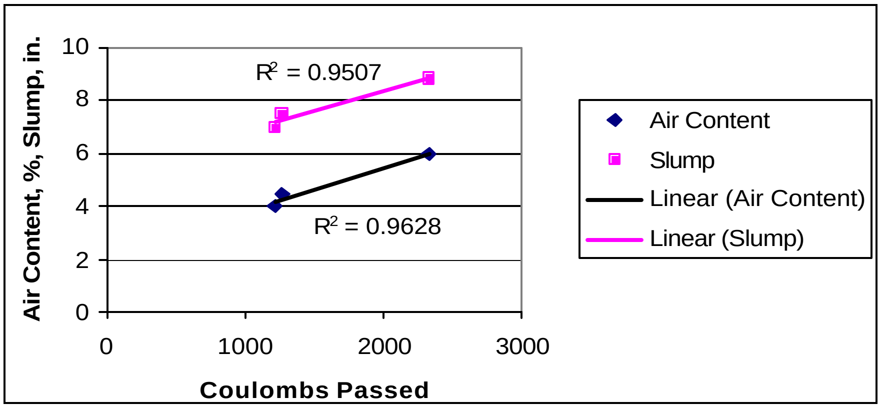


Figure 5-15. Coulombs versus air content and slump for Mix 7

6. FIELD SAMPLING AND SURVEY RESULTS

6.1 Model Verification

The chloride content information collected in this study was used to evaluate the use of diffusion models to predict the chloride concentration of bridge components. It was also used to develop model criteria for bridges in inland regions.

Prior to evaluating the model used for the design of the Virginia Dare Bridge, the efficacy of using Fick’s equations for long term predictions was tested. It is known that in the field, considerable deviation from pure diffusion processes occurs. Testing the model with field data should suggest the degree to which cracking, temperature variations and material quality variations can be accounted for by the use of an apparent diffusion coefficient.

Historic chloride content data was available from three bridges on the coast of North Carolina. Most of this data was generated as part of maintenance reports and did not illustrate the distribution of chloride across several depths. Rather, there was information about chloride content at between one and three critical depths (usually near the depth of the reinforcing steel). To test the model, data collected during the field sampling component of this project was used to estimate diffusion coefficients and surface chloride concentrations. These values were used in the model to make reverse predictions of the chloride content that should have been detected in similar elements during the previous tests. That is, the input to the model consisted of a diffusion coefficient calculated from 2004 data, a surface chloride coefficient estimated from 2004 data, and an elapsed time equal to the age of the bridge when the previous tests were made.

The three bridges with both 2004 data and historic data available that were used to verify the diffusion model were: the Holden Bridge, the Ocean Isle Bridge and the Bonner Bridge. All three of these structures are on the coast of NC.

Holden Bridge

Figures 6-1 and 6-2 show the locations of samples from the footings of the Holden Bridge. To minimize the affect of diffusion from multiple faces of the footing, samples were taken at least 24” from edges of the element. The numbers (referenced with #) refer to the hole number used to identify results in the tables and chloride profile graphs. Table 4-6 lists the chloride content that was found in each sample. There are missing values in Hole 2 at depths 5” and 7” that are likely due to the drill hitting a rebar.

A visual representation of the data presented in Table 4-6 is also given in Figures 6-3a through 6-3d. It can be seen in the graphs that most of the sampling holes display a typical chloride diffusion profile. The 3” depth of hole 1 has a strange profile that was probably caused either by some interaction from a non-diffusion process, micro-cracking, or contamination of the sample.

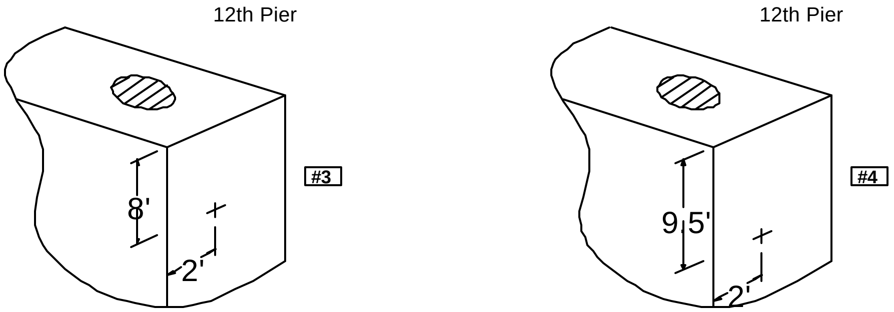


Figure 6-1. Location of samples from Holden Bridge footings at bent 12

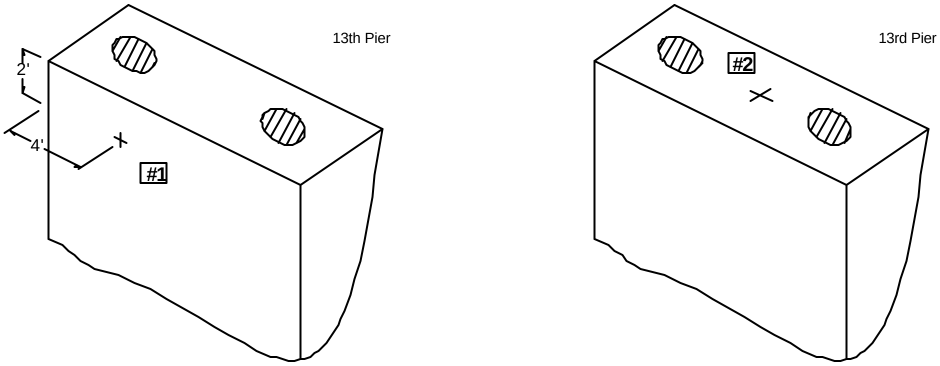
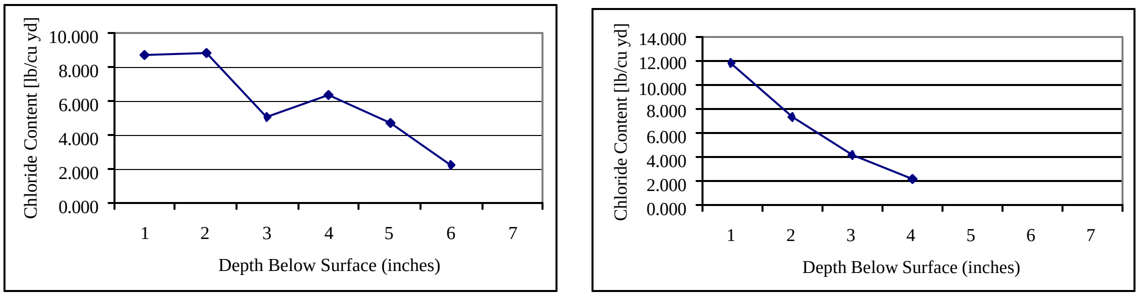


Figure 6-2. Location of samples taken from Holden Bridge footings at bent 13

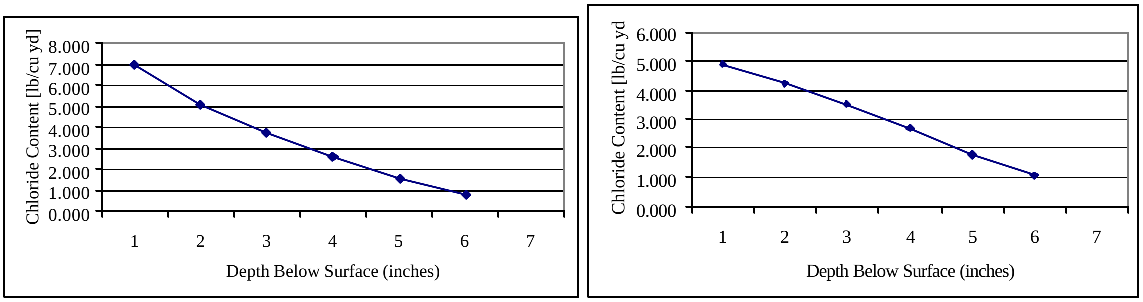
Table 6-1. Chloride content of samples taken from Holden Bridge footings

		Cl Content (lb/yd^3)					
Hole	Sample Location	1"	2"	3"	4"	5"	7"
1	Footing	8.692	8.875	5.121	6.417	4.701	2.214
2	Footing	11.86	7.327	4.199	2.165		
3	Footing	6.953	5.088	3.712	2.558	1.525	0.782
4	Footing	4.876	4.215	3.522	2.699	1.795	1.058



a) Hole 1

b) Hole 2



c) Hole 3

d) Hole 4

Figure 6-3. Chloride content in the footings of Holden Bridge

In order to use the data from the Holden Bridge to verify the diffusion model, some computation was performed on the results. Two computational strategies were used to estimate the diffusion coefficient (D) and surface concentration (C_0) values. Both of these methods utilized the 'genfit' function in MathCAD described earlier. Unlike many other curve fitting routines, 'genfit' is capable of fitting an arbitrary equation to a set of data points. For the case of diffusion, the routine adjusts the D and C_0 values in Fick's formula to match a chloride profile found in the field.

For the information presented in Table 6-1, and using t = 19 years (Holden bridge was built in 1985), the routine has estimated $C_0 = 9.717$ lb/yd^3 and D = 0.491 in^2/year (or $1.003*10^{-11}$ m^2/sec). Although this appears to be a high diffusion coefficient, the distinction between apparent diffusion coefficients and pure diffusion coefficients should explain its magnitude. For the footing, the diffusion coefficient accounts for any in-service cracking that has occurred in the concrete, which would give rise to faster, non-diffusion based ingress. Also, because the footing is in the splash zone, it is likely that sorption is a considerable ingress process. The daily wetting and drying of the footer by the rising tides drives this sorption.

From the output of the curve fitting routine, the specific model formula for the footings of Holden Bridge is Equation 2-2 with the coefficients C_0 and D replaced by the fitted parameters calculated using the MathCAD routine. This specific formula is given by Equation 6-1.

$$C(x,t) = 9.717\left\{1 - erf\left(\frac{x}{2\sqrt{0.491} * t}\right)\right\} \tag{6-1}$$

It should be noted that the curve fitting routine was slightly different from the method proposed by Weyers et al. in the FHWA publication, "Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques" (Weyers et al., 1994), in which method, only D was fitted. To reduce the number of parameters to one, the surface concentration was taken as the chloride content found at 0.5" depth. FHWA research presented in the paper indicated that the chloride concentration at this depth remains relatively stable after 4-6 years in service. The concentration at 0.5" depth does not fluctuate as radically as the concentration on the actual surface.

Initially in the work at UNCC, both the single parameter (FHWA method) and the multi-parameter techniques (presented above) were attempted. As a verification of both the UNCC technique of multiple parameter curve fitting, and the FHWA assumption of chloride concentration stability at 0.5”, the two values were always close to each other. However, in preliminary modeling trials, it was discovered that the coefficients derived by multi-parameter curve fitting provided more accurate projections. For instance, in the Holden Bridge example, the average chloride concentration at 1” depth is 8.1 lb/yd^3. It can be reasonably assumed that the concentration at 0.5” is slightly higher. Therefore, the chloride content at 0.5” is not much different than the calculated C_0 of 9.7 lb/yd^3 made by the MathCAD routine.

To check the applicability of Equation 6-1 to modeling the chloride content of footings on Holden Bridge, the equation was first checked for the error that it produced in predicting the current chloride content. That is, the surface concentration and diffusion coefficients computed by the MathCAD routine were used with a t equal to 19 years, the time at which the bridge footings were sampled for this project in 2004. An accurate model should provide predictions that are very close to the measured chloride content. Table 6-2 is a comparison of the predicted and measured chloride content. The measured chloride content is an average of the chloride content at each depth for all four 2004 sampling locations on the footing. It can be seen that there is very little difference between the average measured chloride content and the chloride content predicted using Equation 6-1.

Table 6-2. Measured versus predicted chloride content in Holden Bridge footings

	Depth					
	1"	2"	3"	4"	5"	7"
Measured Cl Content (lb/yd^3)	8.10	6.38	4.14	3.46	2.67	1.35
Predicted Cl Content (lb/yd^3)	7.94	6.25	4.74	3.44	2.40	1.02
Difference	0.16	0.12	-0.60	0.02	0.27	0.33
% Difference	1.9%	2.0%	-14.4%	0.5%	10.2%	24.4%

The curve fitting method described above is called the minimization of cumulative sum of squared error (cumulative SSE). The input array contained all the data from the Holden Bridge footings and a diffusion curve was fitted to all data simultaneously. A second test of the modeling formula (Equation 6-1) was made by using it to predict the chloride content of the footings in 1993. This prediction was compared with field data taken by NCDOT officials for a report on the use of epoxy coated reinforcing steel (NCDOT Materials and Tests Unit, 1993). To make the prediction, Equation 6-1 was used with t = (1993-1985) = 8 years. The results of this comparison are shown in Table 6-3, which is very good for the 1” depth, but not as reasonable for the 3” depth.

Table 6-3. Comparison of predicted chloride contents in 1993

	Depth	
Holden Bridge Footings	1"	3"
Measured Cl Content (lb/yd^3)	6.84	1.36
Predicted Cl Content (lb/yd^3)	7.01	2.76
Difference	0.17	1.41
% Difference	2.5%	104.0%

Ocean Isle Bridge

The same computational process was used with data from the Ocean Isle Bridge, originally investigated for chloride content in its footings in 1993. The MathCAD curve fitting routine was used to generate D and C_0 values using chloride profiles found in 2004. These values were then used to back-calculate the expected chloride in 1993 and 1986, respectively.

Table 6-4 shows the results the comparison between the chloride content predicted by the model formula and the 1993 chloride tests on the Ocean Isle Bridge. In this case there were historic chloride data from two locations on the footing. The content at each depth in the two holes was averaged to produce the measured chloride content values given in the Table 6-4. It can be seen that the formula was very effective at predicting the chloride content of the 3” and 5” depths.

However, the 1” depth includes some error. This may be due to the range of the two chloride contents measured in 1993. In the 1993 measurements (shown in Table 6-5), there is a significant difference in the chloride content at the 1” depth between holes 1 and 2. The model predicted a chloride content of 6.66 lb/yd^3, which is close to the content in hole 2, 7.25 lb/yd^3. The high value in hole 1 could have been due to a crack in the concrete near the location of hole 1, or contamination of the sample.

Table 6-4. Predicted vs. actual chloride content in footings of the Ocean Isle Bridge

	Depth		
	1"	3"	5"
Measured Chloride Content (lb/yd^3)	10.82	2.18	0.41
Predicted Chloride Content (lb/yd^3)	6.66	2.29	0.47
Difference	4.16	0.11	0.06
% Difference	38.5%	4.9%	15.7%

Table 6-5. Results from 1993 chloride content tests at Ocean Isle Bridge

	Cl Content (lb/yd^3)		
Hole	1"	3"	5"
1	14.39	2.08	0.51
2	7.25	2.28	0.31

Bonner Bridge

Data from the Bonner Bridge was treated in the same way as described above. Information on chloride content was available from a 1986 study of the bridge done prior to the installation of a cathodic protection system. Chloride content measurements were made at multiple sites on three bents at a depth of 2½”. Measurements made as part of the present project were taken adjacent to the 1986 locations for an accurate comparison. The bridge age at the time of the 1986 measurements was 24 years. The chloride profiles from this study were input in the MathCAD routine, and the results were $C_0 = 25.939$ lb/yd^3, and D = 0.131 in^2/year (or 2.676*10^{-12} m^2/sec); a much more reasonable diffusion coefficient.

Table 6-6 shows the measured values and the predicted values for the two holes sampled in 1986. The percent difference is stated for each case to indicate the amount of error occurring in the prediction, which considering all the factors, would be acceptable.

Table 6-6. Predicted chloride vs. measured chloride for the Bonner Bridge

	Depth	
	2.5"	2.5"
Measured Cl Content (lb/yd^3)	6.98	5.88
Predicted Cl Content (lb/yd^3)	8.27	8.27
Difference	1.29	2.39
% Difference	18.5%	40.6%

This evaluation of the modeling technique indicates that it is possible to use the model to make projections of chloride content at a particular depth, after a particular time in service with an expected error of approximately 30% (32.1% was found with the limited number of cases examined in this study). Therefore, the use of the diffusion modeling Equation 2-2 with an apparent coefficient of diffusion and an accurate surface chloride concentration, is a justified method of modeling chloride ingress in cases where 30% error is acceptable.

It must be noted that many of the prediction results had margins of error much greater or much less than 30%. This can be seen in the individual prediction results shown in Tables 4-7 through 4-13. Therefore, the model does not necessarily provide a conservative estimate. There is wide variation in the chloride content of single structural elements that are exposed to nearly identical conditions. The use of estimates from the diffusion model must be with an understanding of this variation.

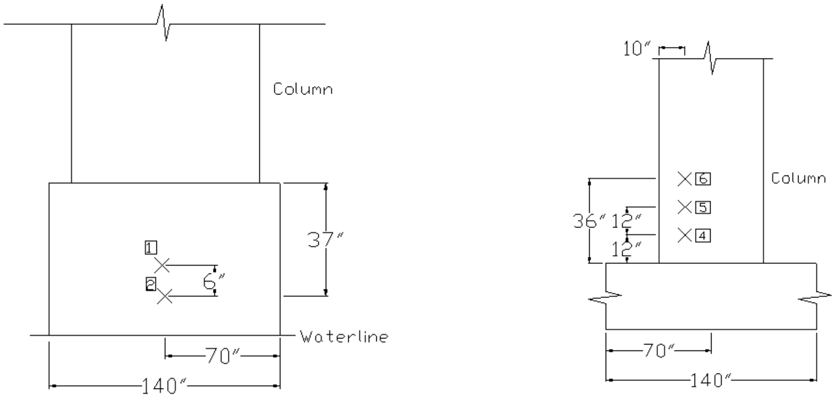
6.2 Virginia Dare Bridge Analysis

The most comprehensive use of the chloride ingress model was made by NCDOT during the design of the Virginia Dare Bridge, completed in 2002. Designers used Equation 2-2 to model the service life of various elements on the bridge. It was desired that each element have 100 year service life. This indicates that the chloride concentration would reach corrosion threshold levels at 95 years. The model was used to determine material characteristics that would provide such a service life. The inputs to the model are given in Table 6-7. The development of these inputs was discussed earlier.

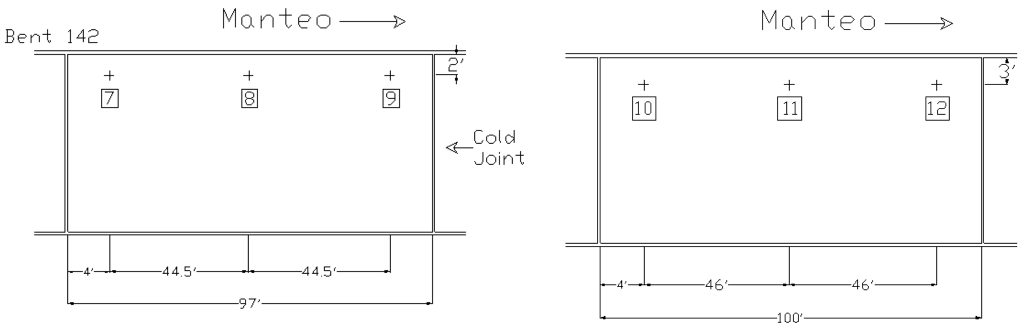
Table 6-7. NCDOT corrosion model inputs

	D (in^2 / yr)	C_0 (lb/yd^3)	k (lb/yd^3)
Deck Slab	0.0783 - 0.147	5.1	0.51
Columns	0.0783 - 0.147	9.9	3.03
Pile Caps	0.0783 - 0.147	19.0	-

It was possible to use data from both the Virginia Dare Bridge and the nearby Bonner Bridge to verify the quantities given in Table 6-7. From the field sampling component of this project, chloride content information was gathered for the footings, piers and deck areas of the Virginia Dare Bridge. The elements sampled are shown in Figures 6-4a through 6-4d. Figure 6-5 is a photograph of the footings that were considered in this evaluation. The chloride content found in these elements is shown in the Table 6-8.



a) b)



c) d)

Figure 6-4. Sampling locations on the Virginia Dare Bridge

Figure 6-5. Footings sampled at the Virginia Dare Bridge

Table 6-8. Virginia Dare Bridge Chloride Content Results

		Cl Content (lb/yd^3)				
Hole	Sample Location	1"	2"	3"	4"	5"
1	Footing	3.493	0.385	0.326	0.327	-
2	Footing	3.97	0.356	0.317	0.319	0.345
3	Footing	2.908	0.368	0.313	0.331	0.343
4	Column	1.101	0.301	0.313	0.325	0.328
5	Column	3.207	0.321	0.332	0.32	0.328
6	Column	0.917	0.308	0.326	0.313	0.315
7	Deck	0.322	0.327	0.315	0.4	0.326
8	Deck	0.316	0.321	0.317	0.322	0.336
9	Deck	0.316	0.357	0.318	0.33	-
10	Deck	0.391	0.359	0.353	0.362	0.354
11	Deck	0.355	0.423	0.396	0.392	0.384
12	Deck	0.356	0.378	0.375	0.386	0.36

Figures 6-6 gives a graphical representation of a typical data shown in Table 6-8. The shape of the chloride profile in the Virginia Dare Bridge is not well defined because the bridge is very new. Most of the deck samples do not have an appreciable amount of chloride in them. The 1” sample depth on the footings has clearly received some chloride from the water in the bay; however, there has not been ample time to permit diffusion to lower depths.

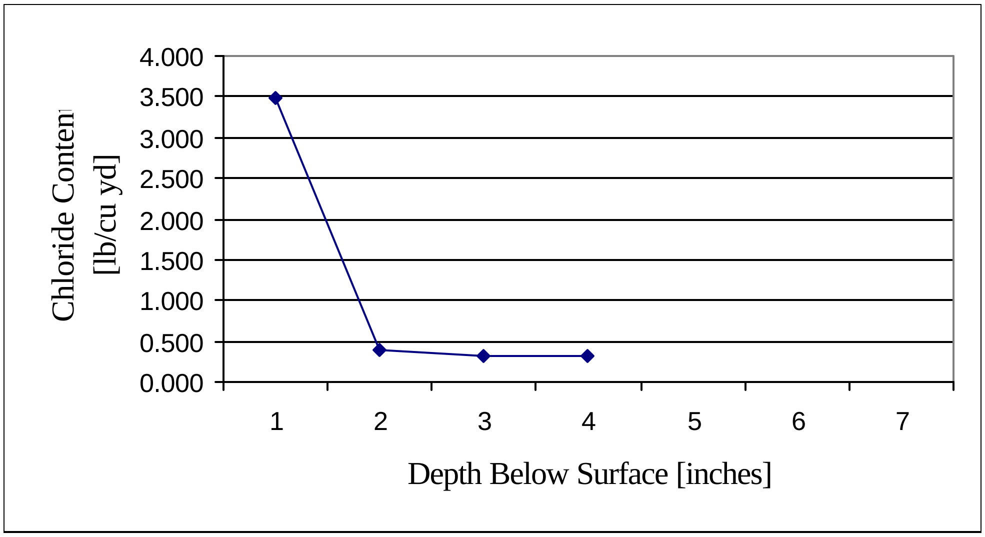


Figure 6-6. Virginia Dare Bridge chloride content – Hole 1

Diffusion Coefficient

The diffusion coefficient, D, is related to several material properties inherent to concrete including the w/c ratio, aggregate type and curing conditions. In order to verify D for the Virginia Dare Bridge the least squares curve-fitting routine described previously was used. Because of the short exposure time for the Virginia Dare Bridge, D could only be computed for only the footing samples. The deck samples contained such small amounts of chloride that the testing procedure was not adequate to develop any resolution in the chloride concentration profile. In the absence of the expected concave chloride profile, it is not possible to estimate a reasonable D value. Three chloride profiles were available for the footings (see Table 6-8).

From Figure 6-6 it is possible to see that there is only one data point that shows a chloride concentration above the background content. Each of the 1" measurements is fairly close to 3 lb/yd^3, the average being 3.16 lb/yd^3.

First, a diffusion coefficient for each hole was calculated individually. These three D values and the associated C_0 were combined to create an average D and an average C_0. To check the accuracy of these values, the error between the chloride content predicted by the model and the chloride content measured in the field was compared in the same way the Holden Bridge model was checked. The comparison is shown in the Table 6-9.

Second, a diffusion coefficient representing data from all three holes was computed. This computation was completed with the cumulative minimization of the squared sum of error method described previously. The D and C_0 values found by this method are shown in the Table 6-10. Their error in predicting the measured chloride content is shown as well.

Table 6-9. Virginia Dare Bridge model verification using averaged C_0 and D

	Depth				
Virginia Dare Bridge Footings	1"	2"	3"	4"	5"
Measured Chloride Content (lb/yd^3)	3.16	0.12	0.08	0.09	0.07
Predicted Chloride Content (lb/yd^3)	3.22	0.13	0.00	0.00	0.00
Difference	-0.06	-0.01	0.08	0.09	0.07
% Difference	-1.9%	-5.5%	98.8%	100.0%	100.0%

C_0= 18.090 lb/yd^3 D= 0.066 in^2/yr

Table 6-10 Virginia Dare Bridge model verification using Cumulative SSE C_0 and D

	Depth				
Virginia Dare Bridge Footings	1"	2"	3"	4"	5"
Measured Chloride Content (lb/yd^3)	3.16	0.12	0.08	0.09	0.07
Predicted Chloride Content (lb/yd^3)	3.15	0.12	0.00	0.00	0.00
Difference	0.01	0.00	0.08	0.09	0.07
% Difference	0.4%	-0.2%	98.9%	100.0%	100.0%

C_0= 17.913 lb/yd^3 D= 0.065 in^2/yr

In the case of the Virginia Dare Bridge, the method of averaging D and C_0 and the method of cumulative error reduction worked nearly equally well. The surface concentration of chloride was estimated at nearly 18.0 lb/yd^3 by both models. This is very close to the model input value of 19 lb/yd^3 used by the NCDOT in designing the bridge. The diffusion coefficient found by both methods was nearly 0.07 in^2/year, which is slightly less than the lowest D value, 0.0783 in^2/year, used in the NCDOT model.

It must be recognized that these verifications are made after the bridge had experienced a very limited amount of time in service, and therefore, a short exposure to chloride. The resolution in the chloride profile was not optimal for the calculation of a diffusion coefficient or of a surface chloride concentration. Despite these shortcomings in the data, there is very good agreement between the material properties and environmental conditions used as inputs in the durability model and those estimated by way of field sampling. Therefore, it is likely that the model-produced projections will accurately estimate the chloride content at the depth of the reinforcing steel after 95 years in service!

Figure 6-7 shows the relationship of bridge age with chloride concentration. The concentration in the model increases to approximately 7 lb/yd^3 after 95 years in service. The model includes a 10 year lateral shift to account for the use of epoxy coated rebars. Thus, if the corrosion threshold is approximately 7 lb/yd^3 for the concrete used in the footers, and duration of time between initiation of corrosion and major maintenance is 5 years, the projection of a 100 year service life is accurate.

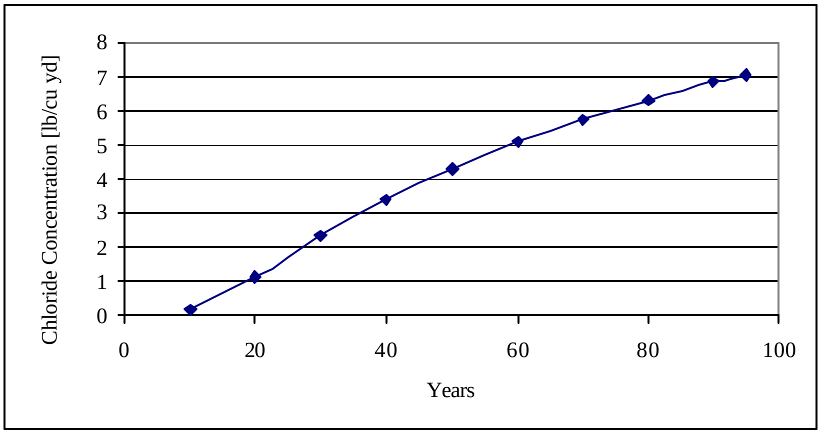


Figure 6-7. Increase of chloride concentration in the Virginia Dare Bridge over 95 years

RCPT Test Results

Six concrete cores were removed from the Manteo Bypass for testing as part of this project (Figure 6-8). The cores came from three sections of the deck and three footings. These were the footings at bents 137, 138 and 139. The three deck spans were 125, 135 and 142. Typically two cores were removed from each of these areas because the dense rebar grid made it difficult to produce specimens without steel contamination.

Two cores were also collected from a test slab that was poured at the same time as the bridge deck. The slab was moved to the resident engineer’s office, and was stored away from the ocean. Specimens were taken from the locations indicated in Table 6-11. The test results are shown in Table 6-12. The cumulative coulombs passed after six hours are provided for each specimen.

Figure 6-8. Collecting core samples from the Virginia Dare Bridge, Manteo, NC

Table 6-11. Origin of RCPT specimens

Specimen ID	Sampling Location
139 B 2 139 A 1 139 A 2 139 B 1	Footing at bridge bent 139
Hole 9 B 1 Hole 9 A 2 Hole 9 B 3 Hole 9 A 3	Deck span 142
135 B 2 135 A 1 135 A 2 135 B 1	Deck span 135
Slab 1-1 Slab 2-2 Slab 1-2 Slab 2-1	Slab stored at the Resident Engineer's office
Near 11 B 1 Near 11 B 2 Near 11 A 1 Near 11 A 2	Deck Span 125
138 A 1 138 B 1 138 A 2 138 B 2	Footing at bridge bent 138
137 A 2 137 B 1 137 A 3 137 A 1	Footing at bridge bent 137

Table 6-12. Coulombs passed after six hours for all specimens

Sample	Coulombs Passed	Average Coulombs Passed	Standard Deviation	Coefficient of Variation
139 B 2	631	630	37.3	5.9
139 A 1	582			
139 A 2	634			
139 B 1	673			
Hole 9 B 1	269	282	48.3	17.1
Hole 9 A 2	238			
Hole 9 B 3	351			
Hole 9 A 3	270			
135 B 2	344	427	195.4	45.8
135 A 1	334			
135 A 2	310			
135 B 1	719			
Slab 1-1	309	293	14.6	5.0
Slab 2-2	300			
Slab 1-2	277			
Slab 2-1	284			
Near 11 B 1	421	372	44.8	12.0
Near 11 B 2	358			
Near 11 A 1	392			
Near 11 A 2	317			
138 A 1	892	1029	627.7	61.0
138 B 1	554			
138 A 2	1947			
138 B 2	721			
137 A 2	570	1106	847.0	76.6
137 B 1	2312			
137 A 3	466			
137 A 1	1077			

It can be seen that there were clear outliers in the results. Often one specimen has registered much higher coulomb value than the others. The test procedure given in ASTM C1202 states that an acceptable coefficient of variation between two specimens of the same concrete is 12.3% (ASTM, 00); thus, results should vary by no more than 35%.

The specimens that were significantly different from the others in the batch typically were found to have flaws.

As an example of the flaws, specimen 135 B1 had a void that penetrated nearly half of its thickness. This allowed current to flow through a section with much less resistance and resulted in a greater amount of charge being passed. In the samples from the footings, the specimens labeled with a 1 (ie. 137 B1) were cut from the uppermost portion of the core. This means that they were closest to the surface of the footing when in-service. It is known from the tests of concrete powder samples from the Virginia Dare bridge that a chloride concentration of up to 3.97 lb/yd^3 was present in the first 1” of this concrete. Because the RCPT is not designed for evaluating concrete that has a significant quantity of chloride already in the material, this may explain the high readings.

In Table 6-13 the results of the RCPT tests are shown again with the outlier specimens removed. Without the obviously flawed specimens many of the tests are much more consistent. However, some groups still show higher than acceptable variation. It is also apparent that the variation between the concrete in three spans of the deck was much less than the variation of the concrete in the footings. The standard deviation of the average coulombs passed for each sampling location was 49 coulombs for the decks and 248 coulombs for the footings. Therefore, some source of variability in either the concrete mixing, materials or placement has affected the chloride permeability as measured by the RCPT. It should be noted that both concretes achieved test results of less than 1000 coulombs passed, which correlates with very low penetrability and was the target value given by the NCDOT for low permeability concrete.

Although both elements are expected to have very low penetrability to chloride ions, the difference in the two is significant. The concrete mix designs (minus non-corrosion related admixtures) for the two materials are shown in Tables 6-14 and 6-15. The reasons for the lower permeability of the deck mix could be related to the different w/c ratio, which is much lower for the deck concrete. Additional cement in concrete is known to lower the permeability.

Table 6-13. RCPT results with outliers removed

Sample	Coulombs Passed	Average Coulombs Passed	Standard Deviation	Coefficient of Variation
139 B 2	631	630	37.3	5.9
139 A 1	582			
139 A 2	634			
139 B 1	673			
Hole 9 B 1	269	259	18.2	7.0
Hole 9 A 2	238			
Hole 9 B 3				
Hole 9 A 3	270			
135 B 2	344	329	17.5	5.3
135 A 1	334			
135 A 2	310			
135 B 1				
Slab 1-1	309	293	14.6	5.0
Slab 2-2	300			
Slab 1-2	277			
Slab 2-1	284			
Near 11 B 1	421	372	44.8	12.0
Near 11 B 2	358			
Near 11 A 1	392			
Near 11 A 2	317			
138 A 1	892	807	120.9	15.0
138 B 1				
138 A 2				
138 B 2	721			
137 A 2	570	318	73.5	23.2
137 B 1				
137 A 3	466			
137 A 1				

Table 6-14. Deck concrete mix design

Material	Quantity
Cement	641 lb/yd^3
Fly Ash	192 lb/yd^3
Fine Aggregate	1205 lb/yd^3
Coarse Aggregate	802 lb/yd^3
Water	36 gal/yd^3
Corrosion Inhibitor	2 gal/yd^3

Table 6-15. Footing concrete mix design

Material	Quantity
Cement	447 lb/yd^3
Fly Ash	207 lb/yd^3
Fine Aggregate	1112 lb/yd^3
Coarse Aggregate	1751 lb/yd^3
Water	27 gal/yd^3
Corrosion Inhibitor	3 gal/yd^3
Silica Fume	35 lb/yd^3

The results of these tests demonstrate that in spite of the prescriptive specification for materials, it is important to monitor the quality of the concrete as it is being placed. After the outlier specimens were removed from the analysis, no concrete tested in this experiment was found to have a penetrability of more than 1000 coulombs passed. However, the variation between the specimens leads to concerns than the penetrability might be increased by material handling in the field.

6.3 Chloride Exposure in Non-coastal Areas

The previous examples of chloride ingress modeling involved the footings of coastal structures. However, data was also gathered from a large number of bridges that are located inland. The source of chloride in these bridges is from road salting. This difference in source means that the loading rate is much more variable, and the members most affected by chloride exposure are the deck and pier caps. Analysis of the data collected from inland bridges was made with the intention of defining chloride exposure criteria for these bridges that will allow modeling and eventual refinement of the chloride mitigation policy for these bridges.

Estimation of C_0 and D for Inland Bridges

The chloride content results found for bridge decks by field sampling were each entered into the MathCAD program developed by UNC Charlotte. Each bridge deck evaluated typically had three sampling locations. Because the cumulative SSE routine has been found to produce more reliable diffusion coefficients and surface chloride estimates, this method was used.

A surface chloride concentration and diffusion coefficient was estimated for each bridge deck in the study except four decks. The Virginia Dare Bridge deck was not sufficiently contaminated to give any curvature in the chloride content curve. The Bennett Road and NC 191 Bridges were similarly under contaminated. Finally, the 421 Bridge deck data was inconsistent and produced a clearly erroneous result. This was probably due to higher than normal interference from non-diffusion related ingress such as entry through cracks. The results of this analysis are given in Table 6-16.

After computing the surface chloride concentration for each bridge, the average surface concentration by region was computed as well. These values are shown in Table 6-17.

Table 6-16. Calculated surface concentration (lb/yd^3) and diffusion coefficients (in^2/yr)

Bridge Name	Area	Age	Co	D
Elk Mountain Road Bridge	Asheville	34	4.555	0.152
Haywood Road Bridge		30	9.477	0.101
Monte Vista Road Bridge		36	1.995	0.09
Hillside Street Bridge	Charlotte	54	1.017	0.214
Hamilton Street Bridge		29	3.514	0.189
Kings/Kennelworth Street Bridge		20	4.337	0.071
Davidson Street Bridge		31	2.155	0.124
Tryon Street Bridge		37	6.095	0.057
Blue Ridge Road Bridge	Raleigh	37	8.456	0.067
Edward's Mill Road Bridge		31	3.993	0.126
Cornwallis Road Bridge		31	5.029	0.037
Alexander Drive over 147 Bridge		42	2.999	0.03
Alexander Drive over I-40 Bridge		38	3.057	0.213
Alexander Dr over Southern RR		37	3.135	0.307
NC 6 Bridge	Greensboro	47	3.59	0.04
Washington Street Bridge		37	11.44	0.152
Elm Street Bridge		37	7.665	0.072
US 311 Bridge		23	1.082	0.268
Robinhood Road Bridge		41	6.787	0.126

Table 6-17. Average surface concentration (lb/yd^3) for non-coastal regions

Area	Average Co
Triad	6.113
Mountain	5.342
Triangle	4.445
Piedmont	3.424

Surprisingly, the Triad region shows the highest surface chloride concentration; followed by the Mountain region, which was expected, considering the heavy use of deicing materials. These two inland areas are the ones targeted by the NCDOT chloride mitigation policy for the addition of fly-ash in bridge deck concrete.

Estimation of ***k*** *Values*

In order to model the chloride ingress on bridge decks and other elements that are exposed to chloride in a cyclic fashion, it is not the surface chloride content that is important, but the ***k*** value. This coefficient represents the rate at which chloride builds up on the surface. Although it was found that the accuracy of models with provisions for a linear increase in surface chloride versus models that assume a constant surface concentration was little different, the non steady-state regime more closely resembles the field condition. Therefore, the surface concentration build-up coefficients were also considered here.

Data from the Tryon Street Bridge in Charlotte will be used to demonstrate the method of calculating ***k*** values for bridge decks. Samples were taken from the bridge deck and pier caps. The location of these samples is shown in Figure 6-9. The results of the chloride content test are shown in Table 6-18. A graphical representation of the results for Hole 3 is given in Figure 6-10.

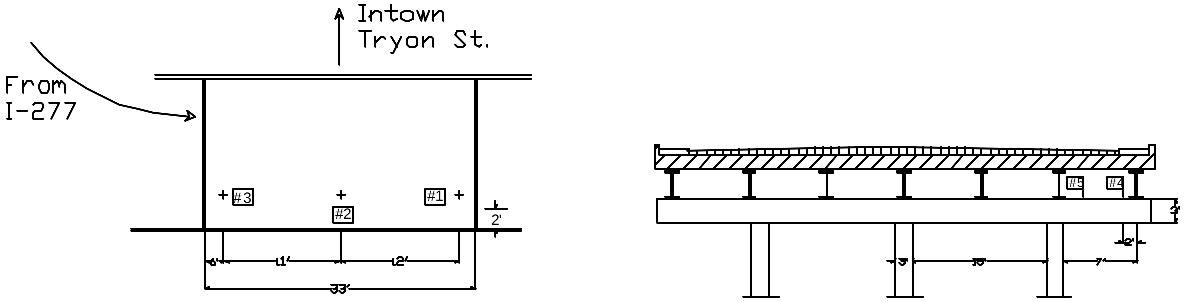


Figure 6-9. Location of Samples taken from the Tryon Street Bridge

Table 6-18. Chloride content of sampled elements on the Tryon Street Bridge

		Cl Content (lb/yd³)				
Hole	Sample Location	1"	2"	3"	4"	5"
1	Deck	3.553	0.826	0.76	0.61	0.292
2	Deck	4.38	1.717	0.853	0.704	0.259
3	Deck	3.903	2.453	1.138	0.648	0.276
4	Pier Cap	4.471	4.117	2.653	1.347	1.232
5	Pier Cap	3.468	2.332	1.632	3.935	1.822

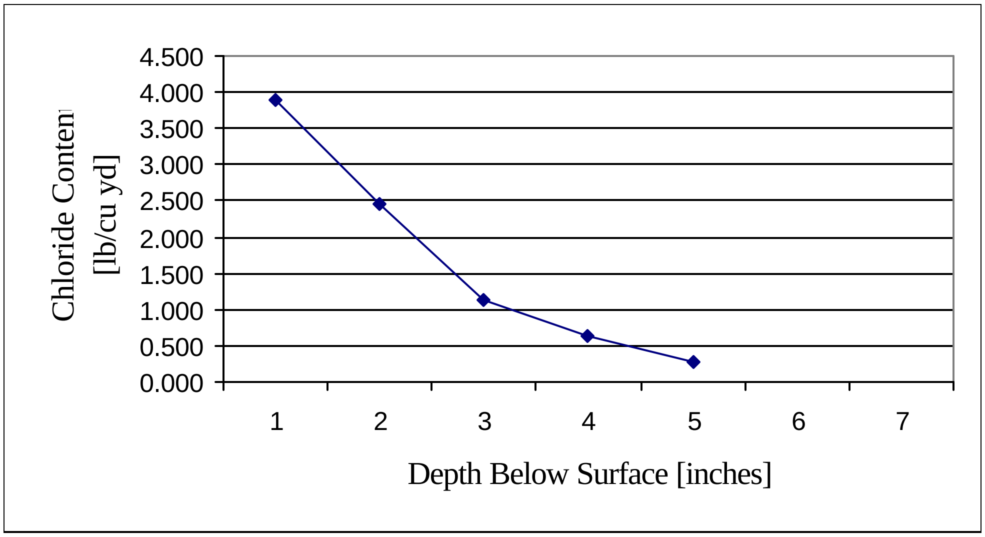


Figure 6-10. Chloride profiles for Tryon Street Bridge – Hole 3

The process of computing ***k*** values is very similar to the process of computing C_0 values. The same MathCAD routine was used to compute these, as well as the same input data. However, the model equation given to the MathCAD routine was different. For ***k***, the model function is Equation 6-2. As with the C_0 routine, the program also requires the input of the partial derivatives of Equation 6-2 with respect to the two fitting parameters, ***k*** and D. The partial derivative with respect to ***k*** is given as Equation 6-3, and the partial derivative with respect to D is given as Equation 6-4. Input was entered into the routine in the form of an array, based on the values given in Table 6-18.

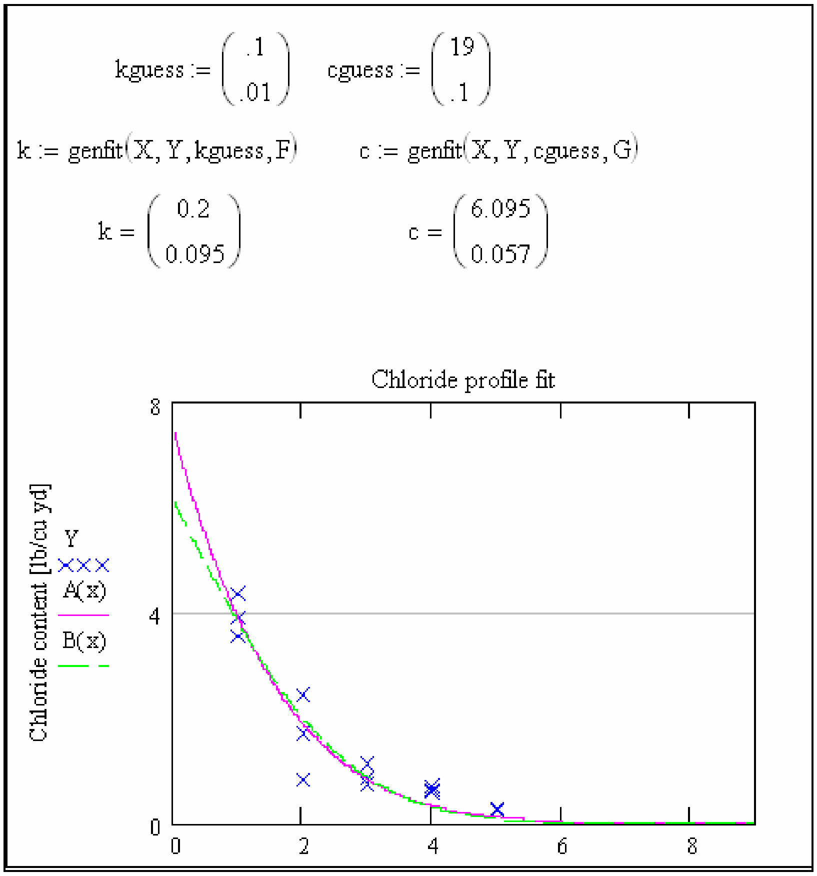
$$C(x,t) = kt\left\{\left(1+\frac{x^2}{2D_c t}\right)erfc\left(\frac{x}{2\sqrt{D_c t}}\right)-\left(\frac{x}{\sqrt{pD_c t}}\right)e^{-x^2/4D_c t}\right\} \tag{6-2}$$

$$\frac{\partial}{\partial k} = 24\cdot\left(1+\frac{1}{48}\cdot\frac{x^2}{D}\right)\cdot\text{erfc}\left[\frac{1}{48}\cdot x\cdot\frac{24^{\frac{1}{2}}}{(D)^{\frac{1}{2}}}\right] - x\cdot\frac{24^{\frac{1}{2}}}{(\pi\cdot D)^{\frac{1}{2}}}\cdot\exp\left(\frac{-1}{96}\cdot\frac{x^2}{D}\right) \tag{6-3}$$

$$\frac{\partial}{\partial D} = 24 \cdot k \cdot \left[\frac{-1}{48} \cdot \frac{x^2}{(D)^2} \cdot \operatorname{erfc}\left[\frac{1}{48} \cdot x \cdot \frac{24^{\frac{1}{2}}}{(D)^{\frac{1}{2}}} \right] + \frac{1}{48} \cdot \frac{1 + \frac{1}{48} \cdot \frac{x^2}{D}}{\pi^{\frac{1}{2}}} \cdot \exp\left(\frac{-1}{96} \cdot \frac{x^2}{D} \right) \cdot x \cdot \frac{24^{\frac{1}{2}}}{(D)^{\frac{3}{2}}} \right] +$$

$$24 \cdot k \cdot \left[\frac{1}{48} \cdot x \cdot \frac{24^{\frac{1}{2}}}{(\pi \cdot D)^{\frac{3}{2}}} \cdot \exp\left(\frac{-1}{96} \cdot \frac{x^2}{D} \right) \cdot \pi - \frac{1}{2304} \cdot x^3 \cdot \frac{24^{\frac{1}{2}}}{(\pi \cdot D)^{\frac{1}{2}} \cdot (D)^2} \cdot \exp\left(\frac{-1}{96} \cdot \frac{x^2}{D} \right) \right] \quad (6\text{-}4)$$

Figure 6-11 shows a screen shot of the output data from the MathCAD routine. For this set of data, the estimated values are shown in the array “k”. $k_{0,0}$ is the surface buildup coefficient, and $k_{0,1}$ is the apparent diffusion coefficient. In this case, k=0.200 lb/yd^3*year, and D=0.095 in^2/yr.



$kguess := \begin{pmatrix} .1 \\ .01 \end{pmatrix} \qquad cguess := \begin{pmatrix} 19 \\ .1 \end{pmatrix}$

$k := genfit(X, Y, kguess, F) \qquad c := genfit(X, Y, cguess, G)$

$k = \begin{pmatrix} 0.2 \\ 0.095 \end{pmatrix} \qquad c = \begin{pmatrix} 6.095 \\ 0.057 \end{pmatrix}$

Figure 6-11. Screen shot of the MathCAD output

In a similar way, ***k*** values and diffusion coefficients were estimated for the other 18 bridge decks as well. These are shown in Table 6-19. The average ***k*** values for each region are shown in Table 6-20.

Table 6-19. Estimated surface build-up (lb/yd^3) and diffusion coefficients (in^2/yr)

Bridge Name	Area	Age	k	D
Elk Mountain Road Bridge	Asheville	34	0.148	0.305
Haywood Road Bridge		30	0.357	0.195
Monte Vista Road Bridge		36	0.063	0.172
Hillside Street Bridge	Charlotte	54	0.02	0.492
Hamilton Street Bridge		29	0.135	0.375
Kings/Kennelworth Street Bridge		20	0.255	0.13
Davidson Street Bridge		31	0.079	0.235
Tryon Street Bridge		37	0.2	0.095
Blue Ridge Road Bridge	Raleigh	37	0.275	0.115
Edward's Mill Road Bridge		31	0.145	0.247
Cornwallis Road Bridge		31	0.197	0.066
Alexander Drive over 147 Bridge		42	0.082	0.058
Alexander Drive over I-40 Bridge		38	0.087	0.455
Alexander Dr over Southern RR		37	0.09	0.691
NC 6 Bridge	Greensboro	47	0.093	0.068
Washington Street Bridge		37	0.346	0.296
Elm Street Bridge		37	0.239	0.132
US 311 Bridge		23	0.051	0.557
Robinhood Road Bridge		41	0.188	0.235

Table 6-20. Average surface build-up coefficients (lb/yd^3) for non-coastal regions

Area	Average k
Asheville	0.19
Greensboro	0.18
Raleigh	0.15
Charlotte	0.14

In the case of the surface build-up coefficient, as anticipated, the mountain region shows the highest average ***k***. Although there is a strong correlation between the surface chloride concentration and ***k***, there are reasons why there is this discrepancy in this case. The bridges tested in Asheville were, as a median, 4 years younger than the Triad bridges. This has allowed less time for the chloride concentration to accumulate.

6.4 Survey results

Much of the information gathered by the survey was qualitative rather than quantitative. In order to present a summary of the responses, the data reduction strategies mentioned previously were used to make the presentation of each question's answer uniform. For instance, several maintenance engineers responded to the question of salt/abrasive mixing ratios with several different ratios that they use. For summary purposes, these different ratios were averaged by county. Full, unedited results are available Tempest (2004).

Some of the responses were more detailed than anticipated, so a data reduction strategy was employed to allow easier comparison of the responses. The primary quantification that was required for this study was the amount of salt applied per lane mile, per year in each region. The yearly loading per lane mile is a function of the salt/abrasive mixing ratio, application rate and number of applications per year. These three quantities were requested in the survey; however, they often had to be adjusted for uniformity in the following ways:

- If deicing chemicals were used with abrasives, the mixing ratio was used to compute the portion that would be salt.
- If chemicals besides NaCl were used as deicers, their weight was similarly proportioned.
- If multiple ratios were given, the one used to compute the local application rate was an average of all the ratios stated.
- Multiple application rates were also averaged.

Finally, the yearly loading was computed as Equation 6-5.

$$L = P * R * Y \tag{6-5}$$

where:

L=loading rate [lb/lane-mile*year],

P= salt/abrasive mixing ratio [lb NaCl/lb mix],

R= application rate [lb mix/lane-mile],

Y= number of applications per year.

One of the first questions on the survey asked was, which deicing materials are used in the county. The response to this question is given in Table 6-21. It can be seen that all regions are using salt and salt brine. The magnesium chloride is only in use in the mountains. To compute the yearly salt application rate, unless mixing ratios were stated for specific materials (as they often were), it was assumed that the chemical being mixed was plain salt (NaCl).

Table 6-22 shows the median number of times roads are salted in each region, per year. In general, the roads in the Bare Pavement System are salted quite a bit more than the roads in the other usage groups. However, it is also apparent from the survey that some roads not in the Bare Pavement System are salted frequently, despite the policy suggestion not to.

The yearly salt application rate was computed as described with Equation 6-5. Only application rates and frequency of application (Table 6-22) for roads in the Bare Pavement System were considered. This is because, as the policy suggests, US and NC routes and other Secondary routes are often not salted. To consider these routes would give falsely low application rates for roads that are routinely salted. The results of this computation are shown in Table 6-23. It can be seen that the mountain and Triad regions have the highest application rates of the six regions.

Table 6-21. Deicing Materials in Use

Region	Salt	Salt Brine	Calcium Chloride	Liquid Calcium Chloride	Magnesium Chloride
Asheville	x	x	x		x
Charlotte	x	x	x		
Raleigh	x	x		x	
Greensboro	x	x	x		
Outer Banks	x	x	x	x	
Coastal	x	x			

Table 6-22. Number of Applications per Year

Region		Bare Pavement	US&NC Routes	Other Secondary Routes
Asheville	# of Responses	6	6	6
	Median	15.0	15.0	8.0
Charlotte	# of Responses	6	3	4
	Median	4.5	0.0	2.3
Raleigh	# of Responses	6	6	5
	Median	4.0	2.0	2.0
Greensboro	# of Responses	5	5	5
	Median	5.0	0.0	0.0
Outer Banks	# of Responses	7	5	5
	Median	3.0	0.0	2.0
Coastal	# of Responses	6	4	5
	Median	3.3	1.3	1.5

Table 6-23. Salt application rate in lb/lane-mile * year

Region		Bare Pavement	US&NC Routes	Other Secondary Routes
Asheville	# of Responses	6	6	6
	Median	1339.3	1339.3	1052.6
Charlotte	# of Responses	7	7	7
	Median	259.3	0.0	0.0
Raleigh	# of Responses	6	6	6
	Median	523.2	81.5	54.3
Greensboro	# of Responses	5	5	5
	Median	1125.0	0.0	0.0
Outer Banks	# of Responses	7	7	7
	Median	750.0	0.0	0.0
Coastal	# of Responses	6	6	6
	Median	15.0	0.0	15.0

The results of the survey do verify the current NCDOT policy. That is, the Mountain and Triad roads are exposed to considerably more salt on an annual basis than roads in other parts of the state; and therefore, bridges in these areas should be specially treated in the chloride mitigation policy. It is also important to note that roads not included in the Bare Pavement System are salted in the mountain region at comparatively high rates. Some secondary roads in the mountains are being salted at rates equivalent to Bare Pavement routes in other regions. This indicates that special attention should even be paid to bridges on less traveled roads.

The information gathered in this survey will be used to verify the results of field sampling. However, as stand-alone data, it is merely qualitative. Naturally, bridge decks will respond differently to the quantity of salt that is spread on them for deicing purposes. A well drained bridge might have a bulk of the road salt washed from its surface by rain before it is able to diffuse into the concrete. Whereas, a poorly drained bridge might drip brine onto other reinforced concrete elements, such as pier caps and piers, through leaky expansion joints.

7. COMPARISON OF FIELD SAMPLING AND SURVEY RESULTS

It is of interest to the NCDOT to put forth durability design guidelines that are specific to regions of the state away from the coast. These will be based on the anticipated chloride exposure of bridges during service. Because the magnitude of exposure is thought to be correlated with climatic conditions, two geographically based research instruments were used to characterize six regions of North Carolina. The first instrument was a survey of highway maintenance engineers with questions regarding the application rate of road salt. The second instrument was a field sampling program for which concrete powder was removed from 28 bridges around the state to determine the chloride content of bridge elements in the different regions.

In order to attempt a correlation of survey results with field sampling results, computations were made on data from bridge decks in each of the six regions. The decks were selected because they should show the most direct correlation between salt application rates and surface chloride concentration. The pier caps should be less directly affected because their exposure is related to the design and functionality of the bridge deck drainage system.

Data from the road salting survey was reduced in the manner described in the previous section. The value that was ultimately estimated from the responses was the application rate in each district as the number of pounds per lane-mile that are spread yearly on roads in the Bare Pavement System. Because it represents the most critical loading situation, an application rate for roads in the bare pavement network was found as an average of all the responding districts located in each of the six geographic areas delineated in this study.

The application rate estimated from the survey and the surface chloride concentration estimated from field sampling were related. The results of this comparison are shown in Figure 7-1. It can be seen that for the Piedmont, Triad and Triangle areas, there is a direct correlation with the stated application rate and the empirically determined surface concentration. The exceptions to the trend are the Coastal bridges and the

Mountain bridges. It is known that the bulk of the loading on the coastal bridge decks comes from sea spray, and so it is not directly tied to road salting. The coastal bridges are not plotted in Figure 7-1 for this reason. It is possible that the bridges sampled in the Asheville area are not routinely salted.

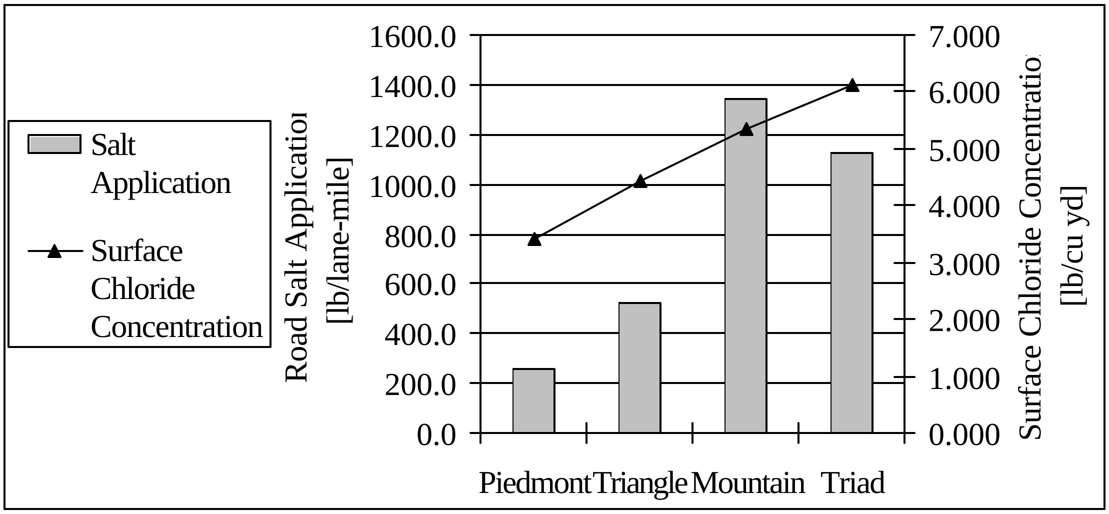


Figure 7-1. Comparison of survey results and field sampling results

It is clear that the surface concentration is directly related to the amount of road salting that occurs in a particular region. The two pieces of information also indicate that the most severe chloride loading is present in the Triad area, followed by the Mountain area, and the Triangle area. The Piedmont area bridges are subject to the least exposure.

In the case of the surface build-up coefficient, the Mountain region shows the highest average ***k***. Although there is a strong correlation between the surface chloride concentration and ***k***, there are reasons why there is this discrepancy in this case. The bridges tested in the Mountain region were, as a median, 4 years younger than the Triad bridges. This has allowed less time for the chloride concentration to accumulate. The discrepancy could also simply be related to the small sample population size. Despite the difference, there is still good agreement between the ***k*** value and the survey data, as is shown in Figure 7-2.

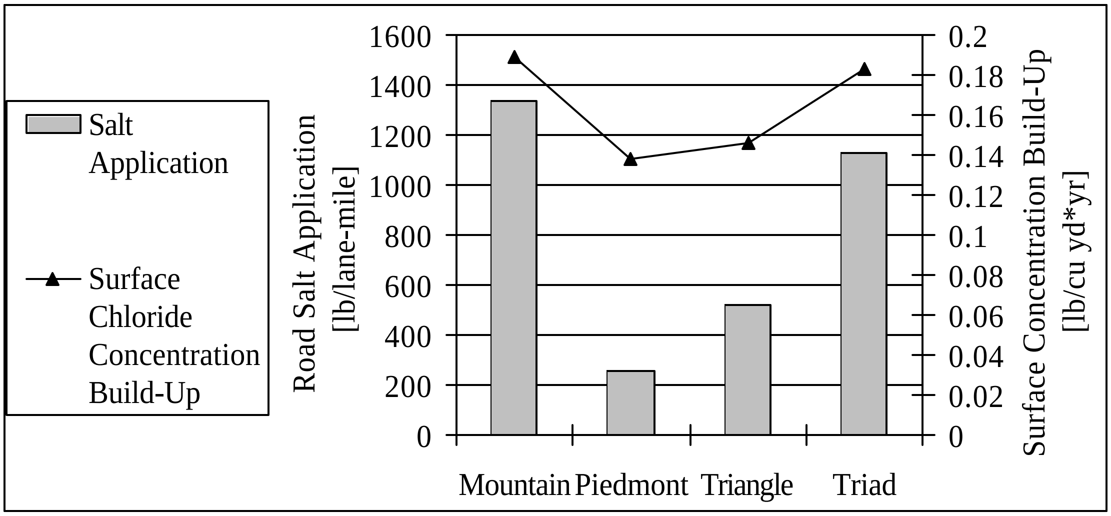


Figure 7-2. Correlation of survey results and k-value from field sampling results

To create a working specification for bridge design in North Carolina, more thorough evaluation of exposure must be completed. However, the process for creating the specification can be illustrated with the results that are available from this study. Assuming no change will be made to road salting practices, the durability design of bridges should be based on the expected surface concentration over time. This involves estimating a surface build-up coefficient. The model has been applied to the design of the Virginia Dare Bridge on the outer banks, and its application to inland bridges would be similar.

Table 7-1 shows the results of such an inland modeling procedure. The design parameters used as inputs to the model were a time till corrosion (T_{corr}) of 70 years and a corrosion threshold of 1.4 lb/yd^3. The 70 year T_{corr} would be applicable to a bridge deck without epoxy coated rebars. It was also assumed that once the corrosion threshold is reached at the level of the steel, there will be five years until corrosion related maintenance will be required. The estimated surface build-up coefficients are used to project a required diffusion coefficient for concrete by solving directly for D.

Table 7-1. Modeling results for 75 year service life of inland bridges

Region	Time (years)	k (lb/ft^3*yr)	D (in^2/yr)	Cover (inch)	Corrosion Threshold (lb/ft^3)
Mountain	70	0.189	0.021	2.0	1.4
Piedmont	70	0.138	0.028	2.0	1.4
Triangle	70	0.146	0.026	2.0	1.4
Triad	70	0.183	0.022	2.0	1.4

The information in the table indicates that in order to prevent the accumulation of corrosion inducing concentrations of chloride at the depth of the steel, it is required that the concrete have the diffusion coefficient listed in the D column. Naturally, there are other mitigation techniques that could be incorporated in the model, such as the use of a corrosion inhibiting admixture or specifying epoxy coated rebars. Both of these strategies would allow the use of more permeable concrete. The economics of bridge construction would dictate the combination of protection schemes that are selected.

Another way to use the data is to make a projection of the remaining service life of the bridges tested. In this case, ***k*** and D from Table 6-19 were used to project the bridge age when the chloride concentration should reach 2 lb/yd^3, the FHWA recommended deck replacement level (NCDOT Materials and Tests Unit, 1993). In the *Current Average Cl at 2”* column, the measured chloride concentration of bridge deck samples at 2” was computed. Many of the bridges have already surpassed the threshold (see Table 7-2).

Of course, more sampling locations per deck would be required to determine a more representative chloride contamination level for the bridge decks. For this study, only three locations along the gutter were sampled. These sites likely have higher chloride concentrations than locations in the lane. However, the data does show that most of the bridges have expected service lives (based on $T_{service}=T_{corr}$) that are significantly shorter than the goal of 75 years set forth by the NCDOT (Rochelle, 2001).

An interesting factor found to affect surface chloride concentration seems to be the amount of traffic carried by a bridge. All bridges with known ADT levels were plotted versus their surface chloride concentrations. This is shown in Figure 7-3. It can be seen that the exposure of bridge decks to chloride is strongly related to the amount of traffic they carry. A few data points were removed from this group because they were strong outliers. The ADT information seemed to be in error in these cases.

Figure 7-3 shows a direct correlation between the amount of traffic carried by a bridge and the surface chloride concentration. This is most likely because the busy bridges are salted more frequently or more heavily. However, the relationship provides the possibility of predicting chloride loading based on expected traffic.

Table 7-2. Estimated service life (years) of existing bridges based on k and D

Bridge Name	Area	Current Age	Time to Reach 2 lb/cu yd	Estimated k	Estimated D	Current Average Cl at 2"
Elk Mountain Road Bridge	1	34	30	0.148	0.305	2.319
Haywood Road Bridge	1	30	20	0.357	0.195	4.406
Monte Vista Road Bridge	1	36	65	0.063	0.172	0.96
Hillside Street Bridge	2	54	133	0.02	0.492	0.663
Hamilton Street Bridge	2	29	30	0.135	0.375	1.65
Kings/Kennelworth Street Bridge	2	20	29	0.255	0.13	0.933
Davidson Street Bridge	2	31	51	0.079	0.235	0.93
Tryon Street Bridge	2	37	38	0.2	0.095	1.665
Blue Ridge Road Bridge	3	37	29	0.275	0.115	2.559
Edward's Mill Road Bridge	3	31	33	0.145	0.247	1.729
Cornwallis Road Bridge	3	31	45	0.197	0.066	0.897
Alexander Drive over 147 Bridge	3	42	79	0.082	0.058	0.462
Alexander Drive over I-40 Bridge	3	38	40	0.087	0.455	1.778
Alexander Dr over Southern RR	3	37	36	0.09	0.691	1.993
NC 6 Bridge	4	47	69	0.093	0.068	0.914
Washington Street Bridge	4	37	17	0.346	0.296	4.579
Elm Street Bridge	4	37	30	0.239	0.132	2.698
US 311 Bridge	4	23	59	0.051	0.557	0.646
Robinhood Road Bridge	4	41	28	0.188	0.235	3.391

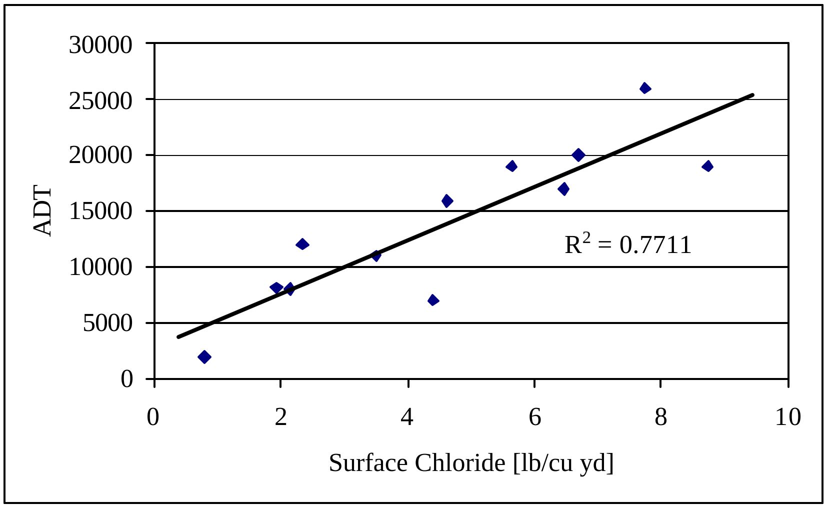


Figure 7-3. Correlation between ADT and surface chloride concentration

As it should be expected, the surface chloride build-up coefficient ***k*** is strongly correlated with the surface concentration, C_0. Because C_0 and the ADT are related, it follows that ADT and ***k*** are related. Figure 7-4, shows this relationship. More research is required to verify the usefulness of this link; however, it appears promising that the expected ***k*** value can be predicted from the expected volume of traffic on the bridge. This suggests that it is possible to use the ADT to estimate a ***k*** value to use in design modeling or in prescriptive specification of chloride mitigation techniques.

There is significant variation between the k values estimated for bridge decks in a specific region. This variation is related to the frequency with which they are salted. Therefore, the most accurate prediction of chloride exposure would include consideration of the ADT, as well as the geographic region. Further research would be required to create a prediction strategy that incorporates these two variables.

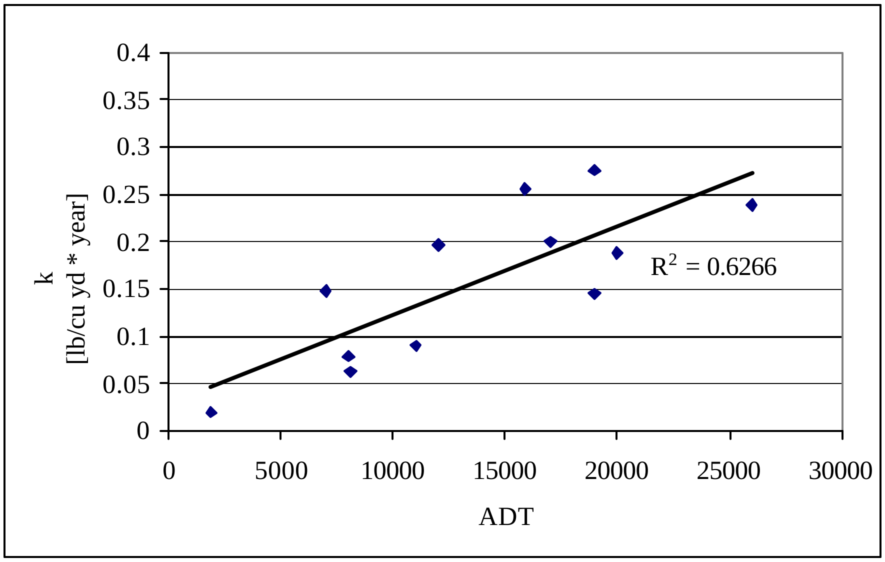


Figure 7-4. Correlation between ADT and surface build-up coefficient ***k***

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Findings and Conclusions

The preset project established, using the ponding test, chloride diffusion coefficients for 10 mix designs widely used in the state of North Carolina. The 10 mixes included: class A concrete from the coast and western part of the state; class AA designs used in bridge decks, columns and footings; and class AA lightweight concrete used in bridge decks. Parallel to this effort, rapid chloride permeability tests (RCPT) have also been performed on all mix designs.

In order to measure the chloride content of the powder samples retrieved during sampling of the ponding slabs, potentiometric titration has been used. However, this procedure is time consuming, and requires a significant effort from laboratory personnel. The original proposal has been amended to include an investigation into the rapid chloride test (RCT), which was found to be a reliable and more practical alternative, and could also be used in the field. As a result, the RCT was used for the vast majority of the powder samples analyzed in this project, with about 10% of the results checked by potentiometric titration.

Through the results of this project it has been shown that there are some reasonable correlations between the salt ponding test and the RCPT results for the ten mix designs tested. The results showed that, when concrete specimens without supplementary cementitious materials are evaluated, the RCPT test results correlate very well with the ponding test, providing a short term estimation of the chloride permeability of the concrete. However, when supplementary cementitious materials and corrosion inhibitor admixtures are added to the mix designs, the correlation becomes much weaker, meaning that the ponding test will need to be completed for the mix designs in question.

During the course of the research, the opportunity arose to test some of the mix designs with two relatively new permeability tests as well. These tests were the ASTM Bulk Diffusion, and NT Build Rapid Migration Test. These two tests provide alternatives to the salt ponding test and the RCPT, respectively. The results obtained from the tests completed provided good correlations with each other, as well as with the ponding and RCPT tests.

It has been also shown that the principles of diffusion can be used to reasonably predict the concentration of chloride in reinforced concrete bridge elements. A model that uses Fick's Second Law of diffusion can estimate the concentration of chloride at the depth of the reinforcing steel after a given amount of time if accurate estimates of the material properties (diffusion coefficient) and environmental conditions (chloride loading) are available. This information can be used with estimates of corrosion threshold to make service life predictions based on corrosion induced deterioration.

It was shown that the modeling procedure used to design the Virginia Dare Bridge is currently valid. Using historic chloride content information, the model was tested for its ability to accurately predict chloride concentration based on information gathered for this research project. These reverse predictions were accurate with a range of error between 20% and 46%.

Field research was conducted in inland areas to determine the necessity of a corrosion design policy for bridges that are subject to road salting as their primary exposure to chlorides. This research phase consisted of results from a survey of road maintenance engineers as well as a field sampling program which tested bridge elements for their chloride content. Results from these studies indicate that there is significant exposure to chlorides in regions across the state.

The surface concentrations reported in this paper should be considered a starting point because there are some shortcomings to the data collection process. Only a limited number of bridges were sampled for this research. There was considerable variation between the chloride loading estimates made for bridges in each area. Although the average loading results do correlate very well with expected results (based on known climate information and road use information), they are not sufficient to make a statistically valid assessment of each region.

The bridges were also concentrated in the major urban areas in each region. For example, all the bridges tested in the Asheville region were located in or around Asheville. Therefore, the results may not be representative of less traveled areas where the roads are salted with less frequency. This is true for all the regions studied.

Unlike bridges on the coast for which the source of chloride is present regardless of the bridge usage, inland bridges are exposed to chloride in a way that is proportional to their use. This was shown by comparing the ADT value with surface chloride concentration and the surface build-up coefficient. Thus, in order to formulate a comprehensive design policy it will be necessary to monitor and to project the amount of road salt that will be applied to a bridge each year. This can be accomplished by considering the ADT as well as the region as indicators of road salt application rates.

The results of the survey verified that the Asheville and Greensboro roads are exposed to considerably more salt on an annual basis than roads in other parts of the state; and therefore, bridges in these areas should be specially treated in the chloride mitigation policy. It is also important to note that roads not included in the Bare Pavement System are salted in the Mountain region at comparatively high rates. Some Secondary Roads in the mountains are being salted at rates equivalent to Bare Pavement routes in other regions! This indicates that special attention should be paid to bridges on less traveled roads as well.

Surprisingly, the Triad region showed the highest surface chloride concentrations when non-coastal bridges were considered, even though the survey suggested a higher deicing salt application rate in the Asheville region. In fact, these values were comparable to some of the coastal bridge results, proving a fairly high chloride contamination of non-coastal bridges. This is also evident from the fact that based on the analyses performed on inland bridge samples, the chloride concentration at 2” depth has already been reached or surpassed the acceptable chloride threshold in half the bridges, suggesting imminent bridge deck replacements at these locations.

Furthermore, the results of inland bridge samples showed that, with only few exceptions, pier caps and abutments have comparable chloride contents (at most of the depths investigated) as bridge decks. This suggests that the chloride mitigation policy should include provisions for these structural elements as well, and provide corrosion protection through special concrete mix designs and rebar types or coatings.

8.2 Recommendations

The research that was conducted yielded some very interesting and useful information, but it also opened new doors and presented new questions. To answer these questions, and to pursue some of the opportunities presented, further research is recommended. Suggestions for the direction of this research are listed below.

- While the ponding test is universally accepted as the test for chloride permeability of concrete, in order to accurately estimate the diffusion coefficient of a concrete mixture, several changes need to be made to the test. ASTM C1543-02 addresses some of these changes, which include epoxying the sides of the slabs, increasing the length of exposure time (which was done in this research project), and profile grinding of the sample, which means that when sampling takes place, the sample is milled in 0.039”-0.079” (1-2 mm) increments.
- The bulk diffusion test is extremely promising and could offer an alternative to the salt ponding test. The advantages of the bulk diffusion test over the ponding test are numerous, and include the need for only casting concrete cylinders instead of slabs, the epoxying of all sides of the specimen except for the exposed surface, and the shorter time period required to obtain meaningful results. Results from the bulk diffusion test can be obtained in as little as 35 days of exposure to the salt solution, with 90 days being more than enough time for even high performance concretes.
- The rapid migration test, which is currently being considered by ASTM, is an extremely attractive alternative to the RCPT test. The RMT addresses some of the problems that plague the RCPT, including the heat build-up issue, the ability to accurately measure the permeability of mixtures containing corrosion inhibitors, and the ability to test specimens which contain steel, so long as the chloride ions do not migrate to the depth of the steel. The only downfall to this test method is the creation of the test equipment. Since the test is not recognized by ASTM or AASHTO yet, there is no standard equipment that can be purchased. The equipment used by the UNCC research team was manufactured in-house, following the design described in Nordtest NT Build 492 (Nordtest, 1999).

- Further research needs to be completed to determine the effect of slump and air content on the permeability of the concrete mix in question. The tests performed at UNC Charlotte indicate that there is some correlation, but this needs to be investigated further.
- A more comprehensive bridge sampling program should be undertaken for inland and coastal structures to develop a state-wide database. This should consist of deck samples from a larger variety of bridges. Because it was found that the chloride content at the 1” depth was fairly indicative of the surface chloride concentration, it is possible that only this depth would be required. By attaining this information, bridge exposure can be classified by ADT and region, thereby defining criteria for future bridges having similar properties. More rapid and economical results will be possible by using the RCT device evaluated in this study. It is important that several samples be taken from each deck in order to give a representative estimation of surface chloride content.
- Although the simple diffusion model was found to be relatively effective at predicting chloride content at discrete depths after a specified exposure time, there are more sophisticated models currently under development. It would be worth investigating these models prior to settling on a research program for development of a more comprehensive mitigation policy. One promising system is the probabilistic model, designed to account for the variation in bridge construction, materials and exposure. Rather than discrete input values, the model represents loading conditions, depth of steel, diffusion coefficients, etc… as statistical distributions. The computation of service life is completed as a statistical re-sampling exercise. The model appears to be promising because it is more capable of accounting for real service conditions. A good treatment of the process of probabilistic modeling is given in Kirkpatrick et al. (2002).
- More work should be done to determine the specific corrosion threshold of concretes used by the NCDOT. This will allow more accurate modeling, and possibly, more economical design. It is known that the addition of fly-ash can reduce the corrosion threshold. Because the mitigation policy requires the addition of fly-ash to some concretes, the effect of this addition should be further investigated.

9. IMPLEMENTATION AND TECHNOLOGY TRANSFER PLAN

Research Product: established chloride diffusion coefficients for 10 mix designs

Suggested User: Materials and Tests, and Structure Design Units

Recommended Use: use these coefficients as a guide for specifying different concrete mix designs to provide adequate corrosion protection.

Recommended Training: none

Research Product: correlation between ponding and RCPT results

Suggested User: Materials and Tests Unit

Recommended Use: for mix designs containing no supplemental cementitious materials, RCPT provides good correlation with ponding test results. For other mixes, consider adopting the RMT and bulk diffusion tests to obtain information on diffusion coefficients in a timelier way.

Recommended Training: none for the RCPT, but minimal training is required for the bulk diffusion test; and more training and test setup manufacturing required for the RMT.

Research Product: potentiometric titration versus RCT method

Suggested User: Materials and Tests, and Bridge Maintenance Units

Recommended Use: use RCT to determine concrete powder chloride content for above mentioned test methods, and for sampled collected from structural components in service.

Recommended Training: minimal, simply following RCT kit manufacturer's recommendations

Research Product: survey on deicing material application

Suggested User: Materials and Tests, Structure Design, and Bridge Maintenance Units, and all divisions

Recommended Use: a refined version of the survey used in this research, combined with a more comprehensive record keeping and monitoring program will improve existing chloride policy.

Recommended Training: none

Research Product: bridge sampling results

Suggested User: Materials and Tests, Structure Design, and Bridge Maintenance Units

Recommended Use: Triad and Mountain regions provided the highest chloride concentrations at the surface and reinforcement levels (levels close to some coastal bridges). In addition, the sampled bridge abutments and pier caps showed comparable chloride contaminations to bridge decks, suggesting that the current chloride mitigation policy should include other structural components for inland bridges.

Recommended Training: none

Research Product: service life prediction model

Suggested User: Materials and Tests, Structure Design, and Bridge Maintenance Units, and all divisions

Recommended Use: with a few bridge concrete samples, it was possible to use the MathCAD routine to reasonably predict the service life (or past performance) of several bridges investigated at the Coastal and Outer Banks regions. More sophisticated models are being developed by others, and those, in combination with the method presented here, could be included in a refined chloride mitigation policy and service life prediction policy. This will allow NCDOT officials to better predict and schedule bridge and bridge deck replacement programs.

Recommended Training: minimal

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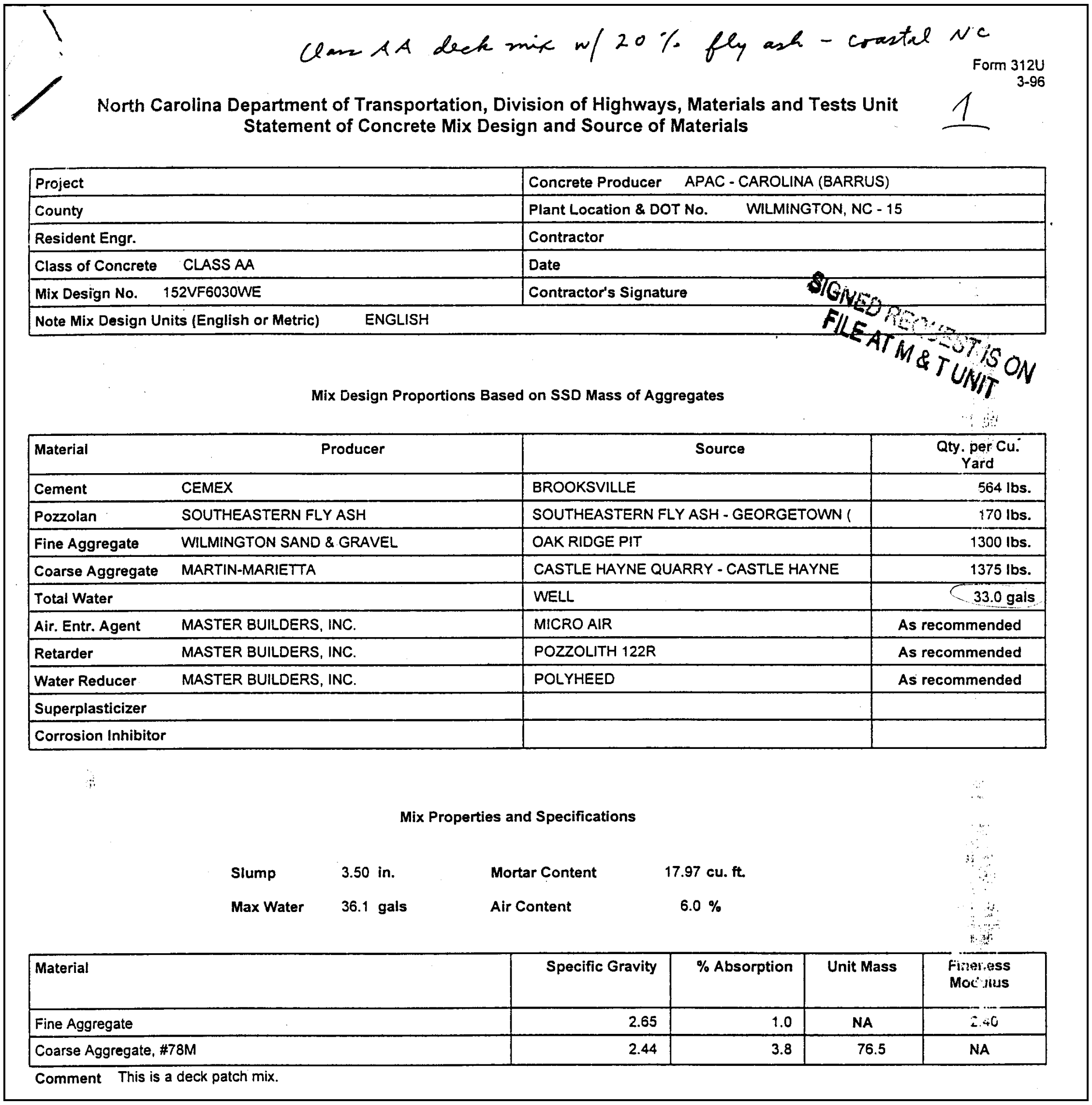
APPENDIX A – CONCRETE MIX DESIGNS



Class AA deck mix w/ 20% fly ash - coastal NC

Form 312U
3-96

1

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Project	Concrete Producer APAC - CAROLINA (BARRUS)
County	Plant Location & DOT No. WILMINGTON, NC - 15
Resident Engr.	Contractor
Class of Concrete CLASS AA	Date
Mix Design No. 152VF6030WE	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

SIGNED REQUEST IS ON FILE AT M & T UNIT

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	CEMEX	BROOKSVILLE	564 lbs.
Pozzolan	SOUTHEASTERN FLY ASH	SOUTHEASTERN FLY ASH - GEORGETOWN (	170 lbs.
Fine Aggregate	WILMINGTON SAND & GRAVEL	OAK RIDGE PIT	1300 lbs.
Coarse Aggregate	MARTIN-MARIETTA	CASTLE HAYNE QUARRY - CASTLE HAYNE	1375 lbs.
Total Water		WELL	33.0 gals
Air. Entr. Agent	MASTER BUILDERS, INC.	MICRO AIR	As recommended
Retarder	MASTER BUILDERS, INC.	POZZOLITH 122R	As recommended
Water Reducer	MASTER BUILDERS, INC.	POLYHEED	As recommended
Superplasticizer			
Corrosion Inhibitor			

Mix Properties and Specifications

Slump	3.50 in.	Mortar Content	17.97 cu. ft.
Max Water	36.1 gals	Air Content	6.0 %

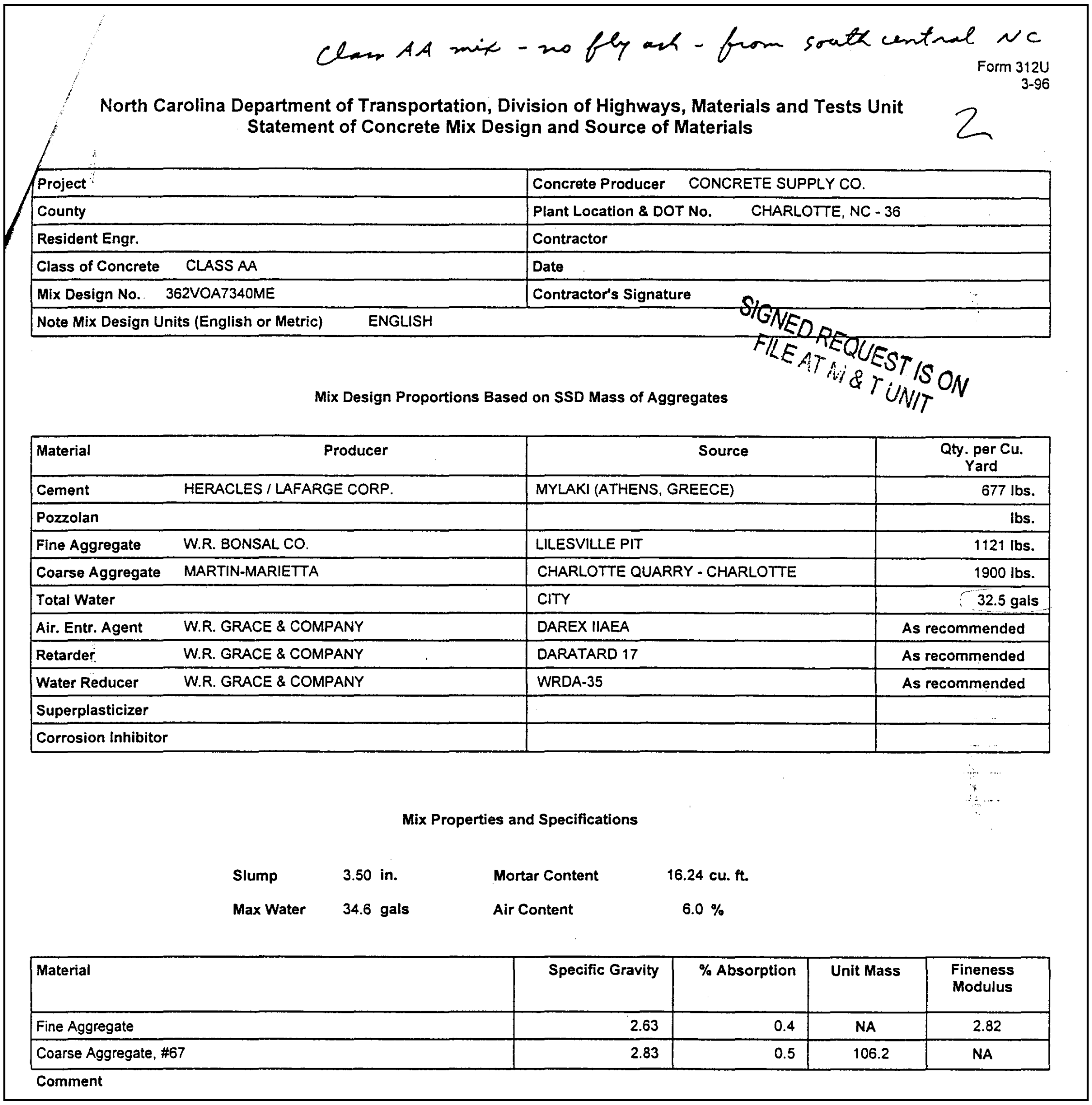
Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.65	1.0	NA	2.40
Coarse Aggregate, #78M	2.44	3.8	76.5	NA

Comment This is a deck patch mix.

Figure A-1. Mix design 1

Table A-1. Mix results for mix design 1

Mix #	Component	Source	Quantity		Test Results			Notes	Notes	
1A	Cement	Cemex	20.9	lb	first slump	2.25	inches	added 5 mL 122R after first mix period. Mix temp was 64 degrees. Dry material measurements are not reliable due to a rock found under the scale	Date Poured	16-Jan
	Pozzolan	Southeastern Fly Ash	6.3	lb	first air	4.5	chase		Batch Size	1
	Fine Aggregate	Wilmington Sand & Gravel	47.7	lb	second slump	3	inches		Mix Type	Class AA, with 20% fly ash, from coastal NC
	# 78M	Martin Marietta	50.9	lb	second air	10	air pot			
	Water	Tap	10.6	lb	third slump					
	Pozzolith 122R	Master Builders	15	mL	third air	10	air pot			
	Polyheed	Master Builders	10	mL						
	Micro Air	Master Builders	15	mL						
1B	Cement	Cemex	20.9	lb	first slump	1.25	inches	added 10 ml of 997 after first mix period. We discovered a rock under the scale that threw off measurements for 1-a and 1-b. Measurements for dry materials in these two mixes are not reliable.	Date Poured	16-Jan
	Pozzolan	Southeastern Fly Ash	6.3	lb	first air	3	%		Batch Size	1
	Fine Aggregate	Wilmington Sand & Gravel	47.7	lb	second slump		inches		Mix Type	Class AA, with 20% fly ash, from coastal NC
	# 78M	Martin Marietta	50.9	lb	second air					
	Water	Tap	10.6	lb	third slump					
	Pozzolith 122R	Master Builders	15	mL	third air		inches			
	Polyheed	Master Builders	10	mL						
	Micro Air	Master Builders	7	mL						
1C	Cement	Cemex	20.9	lb	first slump	2.75	inches	added 2 ml microair after first mix period. Add 2 ml 997 at end	Date Poured	16-Jan
	Pozzolan	Southeastern Fly Ash	6.3	lb	first air	1	%, cha		Batch Size	1
	Fine Aggregate	Wilmington Sand & Gravel	47.7	lb	second slump		inches		Mix Type	Class AA, with 20% fly ash, from coastal NC
	# 78M	Martin Marietta	50.9	lb	second air	5.5	air pot			
	Water	Tap	10.6	lb	third slump					
	Pozzolith 122R	Master Builders	16	mL	third air		inches			
	Polyheed	Master Builders	5	mL						
	Micro Air	Master Builders	0	mL						
1D	Cement	Cemex	20.9	lb	first slump	0	inches	Added 3 ml 997 at break. Possible problem causing low slump was dry aggregate. Aggregate was resoaked and mix redone with 1-c admixture rates.	Date Poured	16-Jan
	Pozzolan	Southeastern Fly Ash	6.3	lb	first air		%		Batch Size	1
	Fine Aggregate	Wilmington Sand & Gravel	47.7	lb	second slump	0.25	inches		Mix Type	Class AA, with 20% fly ash, from coastal NC
	# 78M	Martin Marietta	50.9	lb	second air					
	Water	Tap	10.6	lb	third slump					
	Pozzolith 122R	Master Builders	16	mL	third air		inches			
	Polyheed	Master Builders	6	mL						
	Micro Air	Master Builders	2	mL						
1E	Cement	Cemex	20.9	lb	first slump	3	inches		Date Poured	16-Jan
	Pozzolan	Southeastern Fly Ash	6.3	lb	first air		%		Batch Size	1
	Fine Aggregate	Wilmington Sand & Gravel	47.7	lb	second slump	2.75	inches		Mix Type	Class AA, with 20% fly ash, from coastal NC
	# 78M	Martin Marietta	50.9	lb	second air	6.25	air pot			
	Water	Tap	10.6	lb	third slump					
	Pozzolith 122R	Master Builders	16	mL	third air		inches			
	Polyheed	Master Builders	3	mL						
	Micro Air	Master Builders	2	mL						
1F	Cement	Cemex	47	lb	first slump	2.5	inches	mix temperature was 62 F	Date Poured	16-Jan
	Pozzolan	Southeastern Fly Ash	14.2	lb	first air	6.5	%		Batch Size	2.25
	Fine Aggregate	Wilmington Sand & Gravel	107.3	lb	second slump		inches		Mix Type	Class AA, with 20% fly ash, from coastal NC
	# 78M	Martin Marietta	114.6	lb	second air					
	Water	Tap	23.8	lb	third slump					
	Pozzolith 122R	Master Builders	36	mL	third air		inches			
	Polyheed	Master Builders	7	mL						
	Micro Air	Master Builders	4.5	mL						



Class AA mix - no fly ash - from south central NC

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North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

2

Project	Concrete Producer CONCRETE SUPPLY CO.
County	Plant Location & DOT No. CHARLOTTE, NC - 36
Resident Engr.	Contractor
Class of Concrete CLASS AA	Date
Mix Design No. 362VOA7340ME	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

SIGNED REQUEST IS ON FILE AT M & T UNIT

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	HERACLES / LAFARGE CORP.	MYLAKI (ATHENS, GREECE)	677 lbs.
Pozzolan			lbs.
Fine Aggregate	W.R. BONSAL CO.	LILESVILLE PIT	1121 lbs.
Coarse Aggregate	MARTIN-MARIETTA	CHARLOTTE QUARRY - CHARLOTTE	1900 lbs.
Total Water		CITY	32.5 gals
Air. Entr. Agent	W.R. GRACE & COMPANY	DAREX IIAEA	As recommended
Retarder	W.R. GRACE & COMPANY	DARATARD 17	As recommended
Water Reducer	W.R. GRACE & COMPANY	WRDA-35	As recommended
Superplasticizer			
Corrosion Inhibitor			

Mix Properties and Specifications

Slump	3.50 in.	Mortar Content	16.24 cu. ft.
Max Water	34.6 gals	Air Content	6.0 %

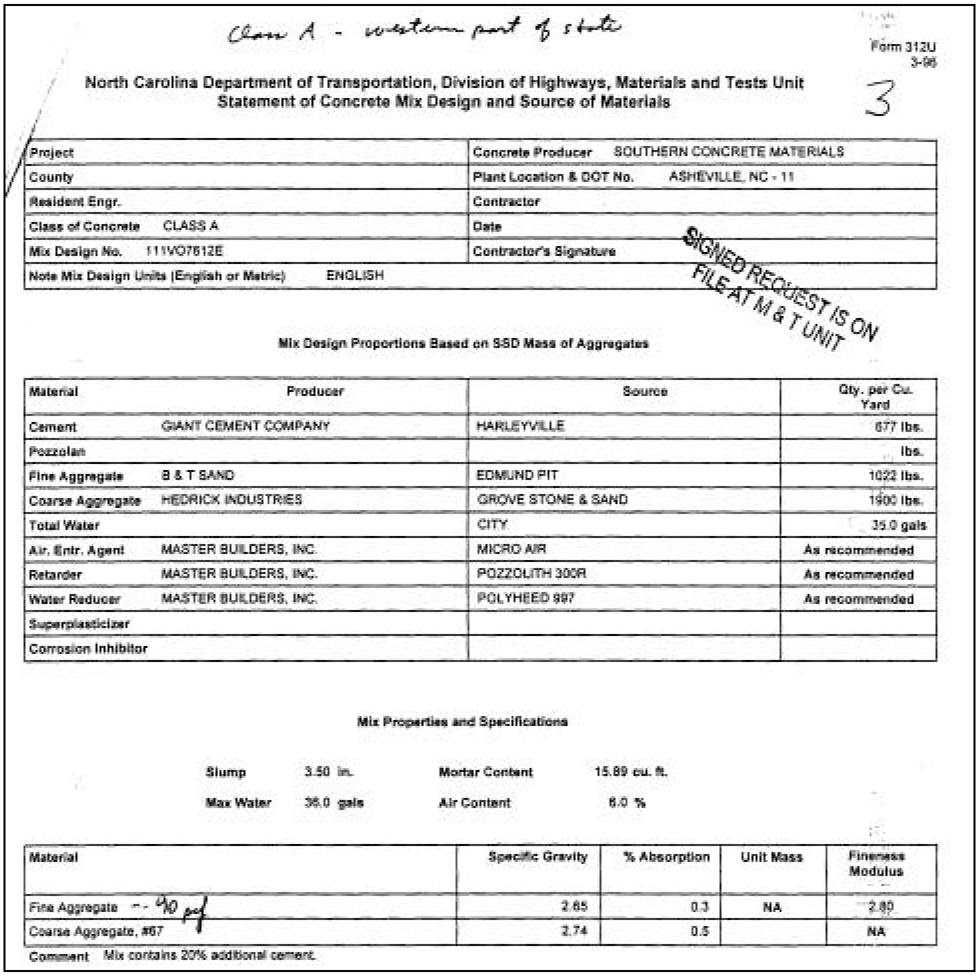
Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.63	0.4	NA	2.82
Coarse Aggregate, #67	2.83	0.5	106.2	NA

Comment

Figure A-2. Mix design 2

Table A-2. Mix results for mix design 2

Mix #	Component	Source	Quantity		Test Results			Notes	Notes	
2A	Cement	Lafarge, Mylaki	62.7	lb	first slump	4.5		with moisture adjustment, add .414 lbs of water. First slump test=4.5", second slump test=4.25". Air test = 8% twice. After air test, we discovered leaky seals on the air pot. We relpaced the seals and read 7% air.	Date Poured	7-Jan
	Fine Aggregate	W.R. Bonsal	104	lb	first air				Batch Size	2.5
	Coarse Aggregate	Martin Marietta	176	lb	second slump	4.25			Mix Type	Class AA, no fly ash, from south central NC
	Water	Tap	25.3	lb	second air					
	Darex IIAEA	W.R. Grace & Company	5	mL	third slump					
	Daratard 17	W.R. Grace & Company	56	mL	third air	7				
	WRDA-35	W.R. Grace & Company	37	mL						
2B	Cement	Lafarge, Mylaki	25.1	lb	first slump	7.5	chase	2-B has wrong amount of water 12.2 lb instead of 10.2 lb.	Date Poured	10-Jan
	Fine Aggregate	W.R. Bonsal	41.1	lb	first air	8			Batch Size	1
	Coarse Aggregate	Martin Marietta	70.4	lb	second slump				Mix Type	Class AA, no fly ash, from south central NC
	Water	Tap	12.2	lb	second air	13	air pot			
	Darex IIAEA	W.R. Grace & Company	2	mL	third slump					
	Daratard 17	W.R. Grace & Company	15	mL	third air					
	WRDA-35	W.R. Grace & Company	15	mL						
2C	Cement	Lafarge, Mylaki	25.1	lb	first slump	9		2-C has wrong amount of water: 12.2 lb instead of 10.2 lb.	Date Poured	10-Jan
	Fine Aggregate	W.R. Bonsal	41.1	lb	first air	8			Batch Size	1
	Coarse Aggregate	Martin Marietta	70.4	lb	second slump				Mix Type	Class AA, no fly ash, from south central NC
	Water	Tap	12.2	lb						
	Darex IIAEA	W.R. Grace & Company	1.5	mL	second air					
	Daratard 17	W.R. Grace & Company	3	mL	third slump					
	WRDA-35	W.R. Grace & Company	3	mL	third air					
2D	Cement	Lafarge, Mylaki	25.1	lb	first slump			added 4 mL IIAEA after first mix period. Mix temp 70 F	Date Poured	10-Jan
	Fine Aggregate	W.R. Bonsal	41.1	lb	first air	3.5	chase		Batch Size	1
	Coarse Aggregate	Martin Marietta	70.4	lb	second slump				Mix Type	Class AA, no fly ash, from south central NC
	Water	Tap	10.2	lb	second air	4	chase			
	Darex IIAEA	W.R. Grace & Company	6	mL	third slump	0.75				
	Daratard 17	W.R. Grace & Company	3	mL	third air	5.8	air pot			
	WRDA-35	W.R. Grace & Company	3	mL						
2E	Cement	Lafarge, Mylaki	25.1	lb	first slump	2.5			Date Poured	10-Jan
	Fine Aggregate	W.R. Bonsal	41.1	lb	first air	6.5	chase		Batch Size	1
	Coarse Aggregate	Martin Marietta	70.4	lb	second slump	2.25			Mix Type	Class AA, no fly ash, from south central NC
	Water	Tap	10.2	lb	second air	7	air pot			
	Darex IIAEA	W.R. Grace & Company	4	mL	third slump					
	Daratard 17	W.R. Grace & Company	3	mL	third air					
	WRDA-35	W.R. Grace & Company	7	mL						
2F	Cement	Lafarge, Mylaki	56.4	lb	first slump	2		mix temp 71 degrees	Date Poured	10-Jan
	Fine Aggregate	W.R. Bonsal	93	lb	first air	6.5	air pot		Batch Size	2.25
	Coarse Aggregate	Martin Marietta	158.3	lb	second slump				Mix Type	Class AA, no fly ash, from south central NC
	Water	Tap	23	lb	second air	7	air pot			
	Darex IIAEA	W.R. Grace & Company	7	mL	third slump					
	Daratard 17	W.R. Grace & Company	7	mL	third air					
	WRDA-35	W.R. Grace & Company	18	mL						



Class A - western part of state

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3

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Project	Concrete Producer SOUTHERN CONCRETE MATERIALS
County	Plant Location & DOT No. ASHEVILLE, NC - 11
Resident Engr.	Contractor
Class of Concrete CLASS A	Date
Mix Design No. 111VO7612E	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

SIGNED REQUEST IS ON FILE AT M & T UNIT

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	GIANT CEMENT COMPANY	HARLEYVILLE	677 lbs.
Pozzolan			lbs.
Fine Aggregate	B & T SAND	EDMUND PIT	1022 lbs.
Coarse Aggregate	HEDRICK INDUSTRIES	GROVE STONE & SAND	1900 lbs.
Total Water		CITY	35.0 gals
Air. Entr. Agent	MASTER BUILDERS, INC.	MICRO AIR	As recommended
Retarder	MASTER BUILDERS, INC.	POZZOLITH 300R	As recommended
Water Reducer	MASTER BUILDERS, INC.	POLYHEED 997	As recommended
Superplasticizer			
Corrosion Inhibitor			

Mix Properties and Specifications

Slump	3.50 in.	Mortar Content	15.89 cu. ft.
Max Water	36.0 gals	Air Content	6.0 %

Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate ~ 90 pcf	2.65	0.3	NA	2.80
Coarse Aggregate, #67	2.74	0.5		NA

Comment Mix contains 20% additional cement.

Figure A-3. Mix design 3

Table A-3. Mix results for mix design 3

Mix #	Component	Source	Quantity		Test Results			Notes	Notes	
3A	Cement	Giant Cement Company	25.1	lb	first slump	6.25			Date Poured	9-Jan
	Fine Aggregate	B&T Sand	37.7	lb	first air	4			Batch Size	1
	Coarse Aggregate	Hedrick Industries	70.4	lb	second slump				Mix Type	Class A, from western part of state
	Water	Tap	10.9	lb	second air					
	Micro Air	Master Builders	1	mL	third slump					
	Pozzolith 300R	Master Builders	30	mL	third air					
	Polyheed 997	Master Builders	23	mL						
3B	Cement	Giant Cement Company	25.1	lb	first slump	1		added .6 mL microair after first air test, and 0.6 mL microair after second air test	Date Poured	9-Jan
	Fine Aggregate	B&T Sand	37.7	lb	first air	2	chase		Batch Size	1
	Coarse Aggregate	Hedrick Industries	70.4	lb	second slump				Mix Type	Class A, from western part of state
	Water	Tap	10.9	lb	second air	2	chase			
	Micro Air	Master Builders	2.4	mL	third slump					
	Pozzolith 300R	Master Builders	37	mL	third air	2.5	air pot			
	Polyheed 997	Master Builders	28	mL						
3C	Cement	Giant Cement Company	25.1	lb	first slump			added 6 ml micro air at first break and 9 ml micro air after second break. Air entrainment didn't increase	Date Poured	9-Jan
	Fine Aggregate	B&T Sand	37.7	lb	first air	3			Batch Size	1
	Coarse Aggregate	Hedrick Industries	70.4	lb	second slump				Mix Type	Class A, from western part of state
	Water	Tap	10.9	lb	second air					
	Micro Air	Master Builders	18	mL	third slump					
	Pozzolith 300R	Master Builders	15	mL	third air					
	Polyheed 997	Master Builders	11	mL						
3D	Cement	Giant Cement Company	25.1	lb	first slump	2	inches	Mix temp was 71 F	Date Poured	9-Jan
	Fine Aggregate	B&T Sand	37.7	lb	first air	7	chase		Batch Size	1
	Coarse Aggregate	Hedrick Industries	70.4	lb	second slump				Mix Type	Class A, from western part of state
	Water	Tap	10.9	lb	second air	7	air pot			
	Micro Air	Master Builders	18	mL	third slump					
	Pozzolith 300R	Master Builders	15	mL	third air					
	Polyheed 997	Master Builders	11	mL						
3E	Cement	Giant Cement Company	56.5	lb	first slump	4"		mix temp ws 70 F	Date Poured	9-Jan
	Fine Aggregate	B&T Sand	84.8	lb	first air	6	chase		Batch Size	2.25
	Coarse Aggregate	Hedrick Industries	158.4	lb	second slump				Mix Type	Class A, from western part of state
	Water	Tap	24.5	lb	second air	7.5	air pot			
	Micro Air	Master Builders	34	mL	third slump					
	Pozzolith 300R	Master Builders	34	mL	third air					
	Polyheed 997	Master Builders	30	mL						

Class AA footings - Manteo Bypass w/ 30% fly ash

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North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

4

Project	Concrete Producer BALFOUR BEATTY CONSTRUCTION COMPANY
County	Plant Location & DOT No. WILMINGTON, NC - 328
Resident Engr.	Contractor
Class of Concrete CLASS AA	Date
Mix Design No. 3282VFHPCMASS1LE	Contractor's Signature
Note Mix Design Units (English or Metric) ENGLISH	

SIGNED REQUEST IS ON FILE AT M & T UNIT

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	LAFARGE CORPORATION	RAVENA	446 lbs.
Pozzolan	BORAL MATERIAL TECHNOLOGIES	PLANT BELEWS CREEK	207 lbs.
Fine Aggregate	PRO SAND CO.	PINEY PROSPECT PIT	1114 lbs.
Coarse Aggregate	HANSON, INC.	ROCKY MOUNT QUARRY	1755 lbs.
Total Water		WELL	27.0 gals
Air. Entr. Agent	MASTER BUILDERS, INC.	MICRO AIR	As recommended
Retarder	MASTER BUILDERS, INC.	POZZOLITH 100XR	As recommended
Water Reducer			
Superplasticizer	MASTER BUILDERS, INC.	REHOBUILD 1000	As recommended
Corrosion Inhibitor	W.R. GRACE & COMPANY	DCI - S	As recommended

Mix Properties and Specifications

Slump	6.00 in.	Mortar Content	16.31 cu. ft.
Max Water	33.5 gals	Air Content	6.0 %

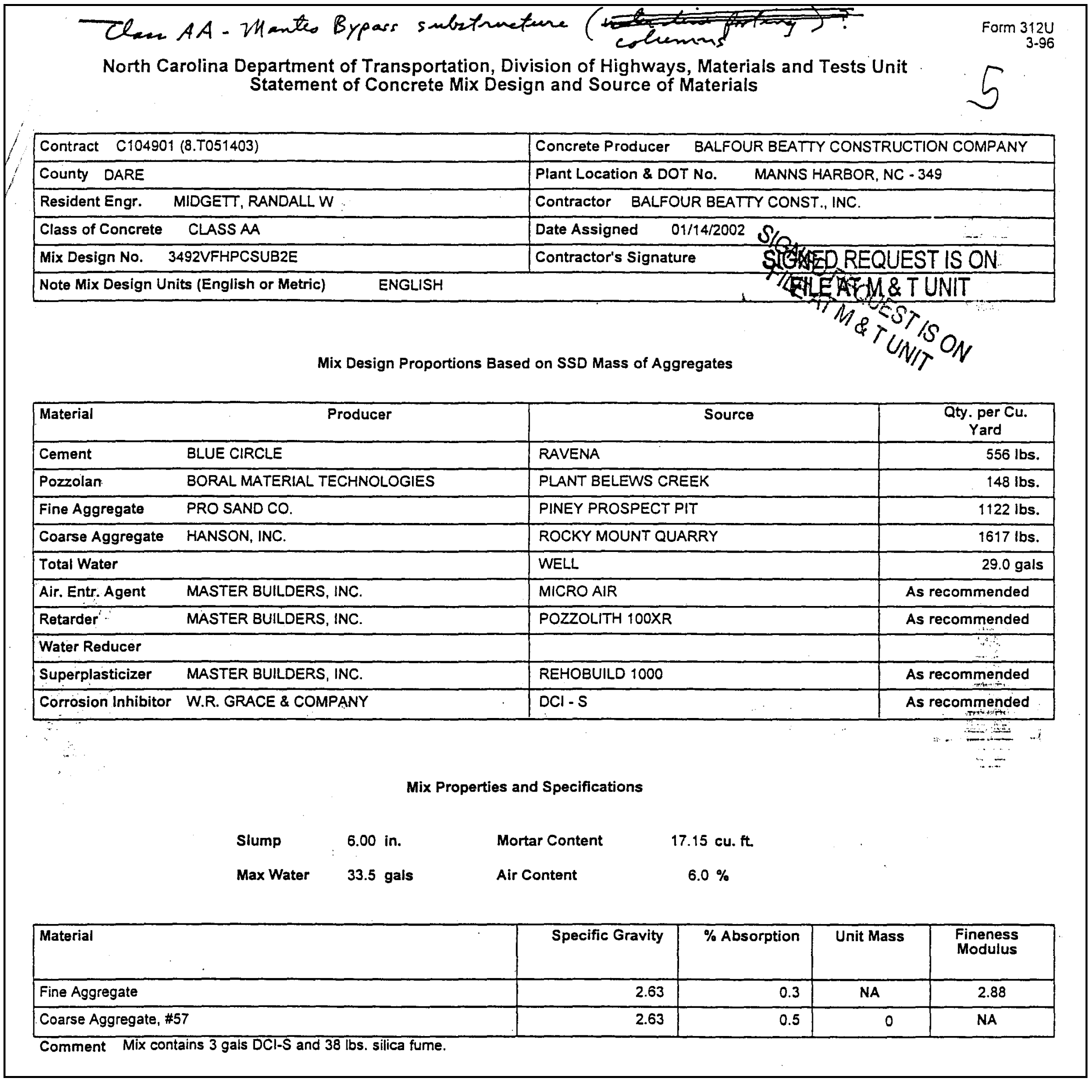
Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.63	0.3	NA	2.88
Coarse Aggregate, #57	2.63	0.5		NA

Comment Mix contains 3 gals DCI-S and 35 lbs. silica fume. This is mass concrete.

Figure A-4. Mix design 4

Table A-4. Mix results for mix design 4

Mix #	Component	Source	Quantity	Test Results	Notes	Notes
4A	Cement	Lafarge, Ravena	24.9 lb	first slump 2.5	added 50 mL R-1000 after first slump test and ran for 2 minutes, added 10 mL R-1000 after second slump test and ran for two minutes.	Date Poured 7-Feb
	Pozzolan	Boral Material Technologies	11.5 lb	first air		Batch Size 1.5
	Fine Aggregate	Pretty Good Sand	61.7 lb	second slump 4.75		Mix Type Class AA footing mix, Manteo Bypass, with 30% fly ash
	# 57	Hanson, Inc	97.6 lb	second air		
	Water	Tap	12.8 lb	third slump 4.25		
	Micro Air	Master Builders	6 mL	third air 2 air pot		
	Pozzolith 100XR	Master Builders	22.1 mL			
	Rehobuild 1000	Master Builders	135 mL			
	DCI-S	W.R. Grace & Company	631 mL			
	Silica fume	Master Builders	1.9 lb			
4B	Cement	Lafarge, Ravena	16.6 lb	first slump 2.5	added 10 ml R-1000 after first slump test, 15 mL R-1000 and 5 mL 100XR after second slump test, 20 mL R-1000 after third slump test.	Date Poured 7-Feb
	Pozzolan	Boral Material Technologies	7.7 lb	first air		Batch Size 1
	Fine Aggregate	Pretty Good Sand	41.1 lb	second slump 2		Mix Type Class AA footing mix, Manteo Bypass, with 30% fly ash
	# 57	Hanson, Inc	65 lb	second air		
	Water	Tap	8.5 lb	third slump 3		
	Micro Air	Master Builders	7 mL	third air 2.5		
	Pozzolith 100XR	Master Builders	15 mL	fourth slump 3.75		
	Rehobuild 1000	Master Builders	128 mL			
	DCI-S	W.R. Grace & Company	421 mL			
	Silica fume	Master Builders	1.3			
4C	Cement	Lafarge, Ravena	20.74 lb	first slump 8		Date Poured 7-Feb
	Pozzolan	Boral Material Technologies	9.58 lb	first air		Batch Size 1.25
	Fine Aggregate	Pretty Good Sand	51.4 lb	second slump 7		Mix Type Class AA footing mix, Manteo Bypass, with 30% fly ash
	# 57	Hanson, Inc	81.3 lb	second air 5 air pot		
	Water	Tap	10.4 lb	third slump		
	Micro Air	Master Builders	12.5 mL	third air		
	Pozzolith 100XR	Master Builders	25 mL			
	Rehobuild 1000	Master Builders	162.5 mL			
	DCI-S	W.R. Grace & Company	526 mL			
	Silica fume	Master Builders	1.6 lb			
4D	Cement	Lafarge, Ravena	33.2 lb	first slump 6		Date Poured 7-Feb
	Pozzolan	Boral Material Technologies	15.4 lb	first air		Batch Size 2
	Fine Aggregate	Pretty Good Sand	82.2 lb	second slump 6		Mix Type Class AA footing mix, Manteo Bypass, with 30% fly ash
	# 57	Hanson, Inc	130 lb	second air 5.5		
	Water	Tap	17 lb	third slump		
	Micro Air	Master Builders	20 mL	third air		
	Pozzolith 100XR	Master Builders	40 mL			
	Rehobuild 1000	Master Builders	260 mL			
	DCI-S	W.R. Grace & Company	842 mL			
	Silica fume	Master Builders	2.6 lb			



Class AA - Manteo Bypass substructure (~~[illegible]~~ columns)

Form 312U
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5

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Contract C104901 (8.T051403)	**Concrete Producer** BALFOUR BEATTY CONSTRUCTION COMPANY
County DARE	**Plant Location & DOT No.** MANNS HARBOR, NC - 349
Resident Engr. MIDGETT, RANDALL W	**Contractor** BALFOUR BEATTY CONST., INC.
Class of Concrete CLASS AA	**Date Assigned** 01/14/2002
Mix Design No. 3492VFHPCSUB2E	**Contractor's Signature** SIGNED REQUEST IS ON FILE AT M & T UNIT
Note Mix Design Units (English or Metric) ENGLISH	

SIGNED REQUEST IS ON FILE AT M & T UNIT

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	BLUE CIRCLE	RAVENA	556 lbs.
Pozzolan	BORAL MATERIAL TECHNOLOGIES	PLANT BELEWS CREEK	148 lbs.
Fine Aggregate	PRO SAND CO.	PINEY PROSPECT PIT	1122 lbs.
Coarse Aggregate	HANSON, INC.	ROCKY MOUNT QUARRY	1617 lbs.
Total Water		WELL	29.0 gals
Air. Entr. Agent	MASTER BUILDERS, INC.	MICRO AIR	As recommended
Retarder	MASTER BUILDERS, INC.	POZZOLITH 100XR	As recommended
Water Reducer			
Superplasticizer	MASTER BUILDERS, INC.	REHOBUILD 1000	As recommended
Corrosion Inhibitor	W.R. GRACE & COMPANY	DCI - S	As recommended

Mix Properties and Specifications

Slump	6.00 in.	**Mortar Content**	17.15 cu. ft.
Max Water	33.5 gals	**Air Content**	6.0 %

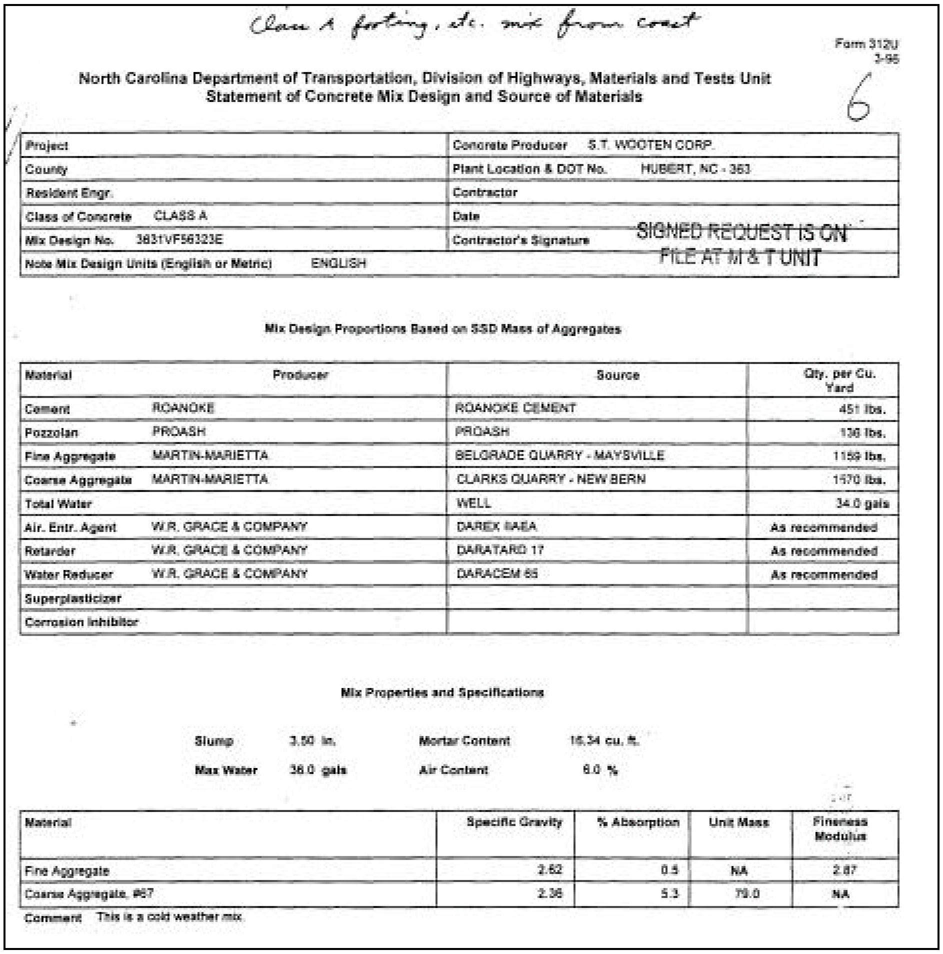
Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.63	0.3	NA	2.88
Coarse Aggregate, #57	2.63	0.5	0	NA

Comment Mix contains 3 gals DCI-S and 38 lbs. silica fume.

Figure A-5. Mix design 5

Table A-5. Mix results for mix design 5

Mix #	Component	Source	Quantity		Test Results			Notes	Notes	
5A	Cement	Lafarge, Ravena	30.9	lb	first slump	5		This mix accidentally did not include silica fume. Added 20 ml R1000 at break.	Date Poured	9-Feb
	Pozzolan	Boral Material Technologies	8.2	lb	first air				Batch Size	1.5
	Fine Aggregate	Pretty Good Sand	62.3	lb	second slump	3			Mix Type	Class AA, Manteo Bypass substructure columns
	# 57	Hanson, Inc	89.8	lb	second air	4				
	Water	Tap	13.6	lb	third slump					
	Micro Air	Master Builders	12	mL	third air					
	Pozzolith 100XR	Master Builders	30	mL						
	Rehobuild 1000	Master Builders	120	mL						
	DCI-S	W.R. Grace & Company	631	mL						
	Silica fume	Master Builders	2.1	lb						
5B	Cement	Lafarge, Ravena	20.6	lb	first slump	4.25		added 15 mL R-1000 after first slump test, 25 mL after second slump test. There was not enough concrete to make two full slabs. Slab 5-B* includes concrete from the air pot in the upper 1".	Date Poured	9-Feb
	Pozzolan	Boral Material Technologies	5.5	lb	first air				Batch Size	1
	Fine Aggregate	Pretty Good Sand	41.6	lb	second slump	4.25			Mix Type	Class AA, Manteo Bypass substructure columns
	# 57	Hanson, Inc	59.9	lb	second air					
	Water	Tap	9.1	lb	third slump	6.25				
	Micro Air	Master Builders	10	mL	third air	3	air pot			
	Pozzolith 100XR	Master Builders	15	mL						
	Rehobuild 1000	Master Builders	140	mL						
	DCI-S	W.R. Grace & Company	421	mL						
	Silica fume	Master Builders	1.4	lb						
5C	Cement	Lafarge, Ravena	20.6	lb	first slump	4.5		added 25 ml R-1000 after first slump test, added 25 mL R-1000 after second slump test. Mix temperature was 65 degrees	Date Poured	9-Feb
	Pozzolan	Boral Material Technologies	5.5	lb	first air				Batch Size	1
	Fine Aggregate	Pretty Good Sand	41.6	lb	second slump	4			Mix Type	Class AA, Manteo Bypass substructure columns
	# 57	Hanson, Inc	59.9	lb	second air					
	Water	Tap	9.1	lb	third slump	5				
	Micro Air	Master Builders	17	mL	third air	4	air pot			
	Pozzolith 100XR	Master Builders	15	mL						
	Rehobuild 1000	Master Builders	175	mL						
	DCI-S	W.R. Grace & Company	421	mL						
	Silica fume	Master Builders	1.4	lb						
5D	Cement	Lafarge, Ravena	20.6	lb	first slump	8			Date Poured	9-Feb
	Pozzolan	Boral Material Technologies	5.5	lb	first air				Batch Size	1
	Fine Aggregate	Pretty Good Sand	41.6	lb	second slump	7.38			Mix Type	Class AA, Manteo Bypass substructure columns
	# 57	Hanson, Inc	59.9	lb	second air	7	air pot			
	Water	Tap	9.1	lb	third slump					
	Micro Air	Master Builders	30	mL	third air					
	Pozzolith 100XR	Master Builders	20	mL						
	Rehobuild 1000	Master Builders	175	mL						
	DCI-S	W.R. Grace & Company	421	mL						
	Silica fume	Master Builders	1.4	lb						
5E	Cement	Lafarge, Ravena	51.5	lb	first slump	8		discovered a tear in the seal of the air-pot after the first air test. The seal was repaired and the second test read 7%. The third test read 8%, however, we could see that air was escaping through the tear. The mix proportions were identical to those in 5D, which registered 7%. The mix temperature was 80 F, this is probably why the slump increased from the mix 5D.	Date Poured	9-Feb
	Pozzolan	Boral Material Technologies	13.7	lb	first air	9			Batch Size	2.5
	Fine Aggregate	Pretty Good Sand	103.9	lb	second slump	7.75			Mix Type	Class AA, Manteo Bypass substructure columns
	# 57	Hanson, Inc	149.7	lb	second air	7				
	Water	Tap	22.9	lb	third slump					
	Micro Air	Master Builders	70	mL	third air	8				
	Pozzolith 100XR	Master Builders	50	mL						
	Rehobuild 1000	Master Builders	413	mL						
	DCI-S	W.R. Grace & Company	1052	mL						
	Silica fume	Master Builders	3.5	lb						



Class A footing, etc. mix from coast

Form 312U
3-95

6

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Project	Concrete Producer S.T. WOOTEN CORP.
County	Plant Location & DOT No. HUBERT, NC - 363
Resident Engr.	Contractor
Class of Concrete CLASS A	Date
Mix Design No. 3831VF56323E	Contractor's Signature SIGNED REQUEST IS ON FILE AT M & T UNIT
Note Mix Design Units (English or Metric) ENGLISH	

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	ROANOKE	ROANOKE CEMENT	451 lbs.
Pozzolan	PROASH	PROASH	136 lbs.
Fine Aggregate	MARTIN-MARIETTA	BELGRADE QUARRY - MAYSVILLE	1159 lbs.
Coarse Aggregate	MARTIN-MARIETTA	CLARKS QUARRY - NEW BERN	1570 lbs.
Total Water		WELL	34.0 gals
Air. Entr. Agent	W.R. GRACE & COMPANY	DAREX IIAEA	As recommended
Retarder	W.R. GRACE & COMPANY	DARATARD 17	As recommended
Water Reducer	W.R. GRACE & COMPANY	DARACEM 65	As recommended
Superplasticizer			
Corrosion Inhibitor			

Mix Properties and Specifications

Slump	3.50 in.	Mortar Content	16.34 cu. ft.
Max Water	38.0 gals	Air Content	6.0 %

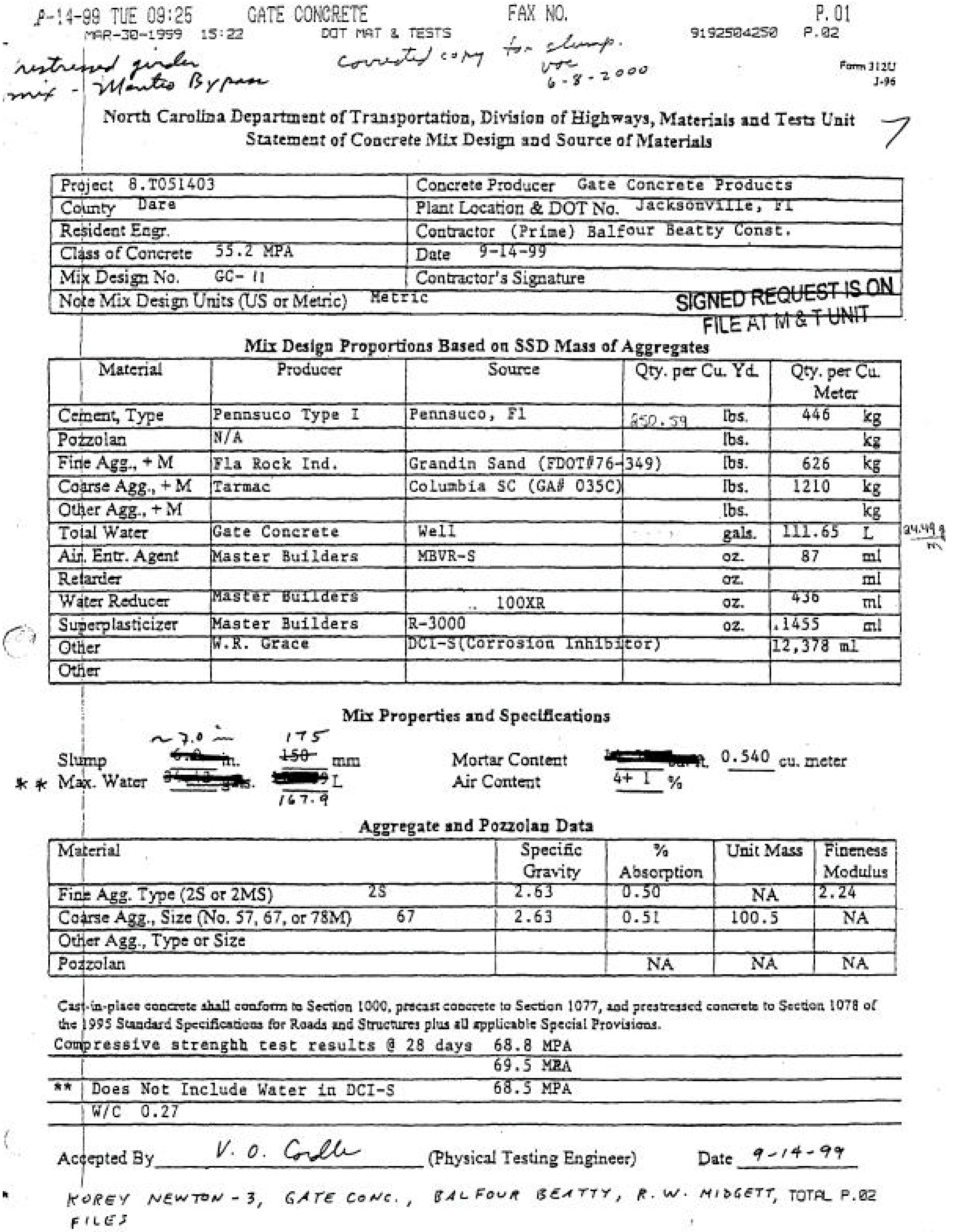
Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.62	0.5	NA	2.87
Coarse Aggregate, #67	2.36	5.3	79.0	NA

Comment This is a cold weather mix.

Figure A-6. Mix design 6

Table A-6. Mix results for mix design 6

Mix #	Component	Source	Quantity		Test Results			Notes	Notes	
6A	Cement	Roanoke Cement	16.7	lb	first slump	2.25		added 4 mL IIAEA after first mix period	Date Poured	13-Jan
	Pozzolan	Proash	5	lb	first air	1.5	chase		Batch Size	
	Fine Aggregate	Martin Marietta, belgrade quarry	42.7	lb	second slump	2.5			Mix Type	Class A footing, mix from coast
	# 67	Martin Marietta, clarks quarry	58.1	lb	second air	4.5	air pot			
	Water	Tap	10.7	lb	third slump					
	Darex IIAEA	W.R. Grace & Company	7	mL	third air					
	Daratard 17	W.R. Grace & Company	3	mL						
	Daracem 65	W.R. Grace & Company	8	mL						
6B	Cement	Roanoke Cement	16.7	lb	first slump	6.5		mix temp 68 degrees F	Date Poured	13-Jan
	Pozzolan	Proash	5	lb	first air	2.5	chase		Batch Size	1
	Fine Aggregate	Martin Marietta, belgrade quarry	42.7	lb	second slump	5			Mix Type	Class A footing, mix from coast
	# 67	Martin Marietta, clarks quarry	58.1	lb	second air	1	chase			
	Water	Tap	10.7	lb	third slump					
	Darex IIAEA	W.R. Grace & Company	10	mL	third air	4.5	air pot			
	Daratard 17	W.R. Grace & Company	3	mL						
	Daracem 65	W.R. Grace & Company	10	mL						
6C	Cement	Roanoke Cement	16.7	lb	first slump	7		mix temp was 63 degrees f	Date Poured	13-Jan
	Pozzolan	Proash	5	lb	first air	1.5	chase		Batch Size	1
	Fine Aggregate	Martin Marietta, belgrade quarry	42.7	lb	second slump	5.25			Mix Type	Class A footing, mix from coast
	# 67	Martin Marietta, clarks quarry	58.1	lb	second air	5.5				
	Water	Tap	10.7	lb	third slump					
	Darex IIAEA	W.R. Grace & Company	10	mL	third air					
	Daratard 17	W.R. Grace & Company	3	mL						
	Daracem 65	W.R. Grace & Company	7	mL						
6D	Cement	Roanoke Cement	16.7	lb	first slump	1.25		added 2 ml Darachem after first mixing period. Added 6 ml IIAEA after second mixing period.	Date Poured	13-Jan
	Pozzolan	Proash	5	lb	first air	2.5	chase		Batch Size	1
	Fine Aggregate	Martin Marietta, belgrade quarry	42.7	lb	second slump	1.5			Mix Type	Class A footing, mix from coast
	# 67	Martin Marietta, clarks quarry	58.1	lb	second air	1	chase			
	Water	Tap	10.7	lb	third slump	2.5				
	Darex IIAEA	W.R. Grace & Company	13	mL	third air	5.5	air pot			
	Daratard 17	W.R. Grace & Company	2	mL						
	Daracem 65	W.R. Grace & Company	4	mL						
6E	Cement	Roanoke Cement	37.6	lb	first slump	1		added another 9 mL of Darachem at break	Date Poured	13-Jan
	Pozzolan	Proash	11.3	lb	first air				Batch Size	2.25
	Fine Aggregate	Martin Marietta, belgrade quarry	96.1	lb	second slump	2			Mix Type	Class A footing, mix from coast
	# 67	Martin Marietta, clarks quarry	130.8	lb	second air	5				
	Water	Tap	24.1	lb	third slump	2.5				
	Darex IIAEA	W.R. Grace & Company	38	mL	third air	5				
	Daratard 17	W.R. Grace & Company	5	mL						
	Daracem 65	W.R. Grace & Company	18	mL						



P-14-99 TUE 09:25 GATE CONCRETE FAX NO. P.01
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restressed girder mix - Manteo Bypass

corrected copy for slump. VOC 6-8-2000

Form 312U
J-96

7

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Project 8.T051403	Concrete Producer Gate Concrete Products
County Dare	Plant Location & DOT No. Jacksonville, Fl
Resident Engr.	Contractor (Prime) Balfour Beatty Const.
Class of Concrete 55.2 MPA	Date 9-14-99
Mix Design No. GC- 11	Contractor's Signature
Note Mix Design Units (US or Metric) Metric	SIGNED REQUEST IS ON FILE AT M & T UNIT

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yd.		Qty. per Cu. Meter	
Cement, Type	Pennsuco Type I	Pennsuco, Fl	850.59	lbs.	446	kg
Pozzolan	N/A			lbs.		kg
Fine Agg., + M	Fla Rock Ind.	Grandin Sand (FDOT#76-349)		lbs.	626	kg
Coarse Agg., + M	Tarmac	Columbia SC (GA# 035C)		lbs.	1210	kg
Other Agg., + M				lbs.		kg
Total Water	Gate Concrete	Well		gals.	111.65	L
Air. Entr. Agent	Master Builders	MBVR-S		oz.	87	ml
Retarder				oz.		ml
Water Reducer	Master Builders	100XR		oz.	436	ml
Superplasticizer	Master Builders	R-3000		oz.	.1455	ml
Other	W.R. Grace	DCI-S(Corrosion Inhibitor)			12,378 ml	
Other						

24.49 g / m

Mix Properties and Specifications

Slump ~7.0 in [redacted] in. 175 ~~150~~ mm — Mortar Content [redacted] 0.540 cu. meter

** Max. Water [redacted] gals. [redacted] 167.9 L — Air Content 4+ 1 %

Aggregate and Pozzolan Data

Material		Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Agg. Type (2S or 2MS)	2S	2.63	0.50	NA	2.24
Coarse Agg., Size (No. 57, 67, or 78M)	67	2.63	0.51	100.5	NA
Other Agg., Type or Size					
Pozzolan			NA	NA	NA

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the 1995 Standard Specifications for Roads and Structures plus all applicable Special Provisions.

Compressive strenghh test results @ 28 days 68.8 MPA
69.5 MPA
** Does Not Include Water in DCI-S 68.5 MPA
W/C 0.27

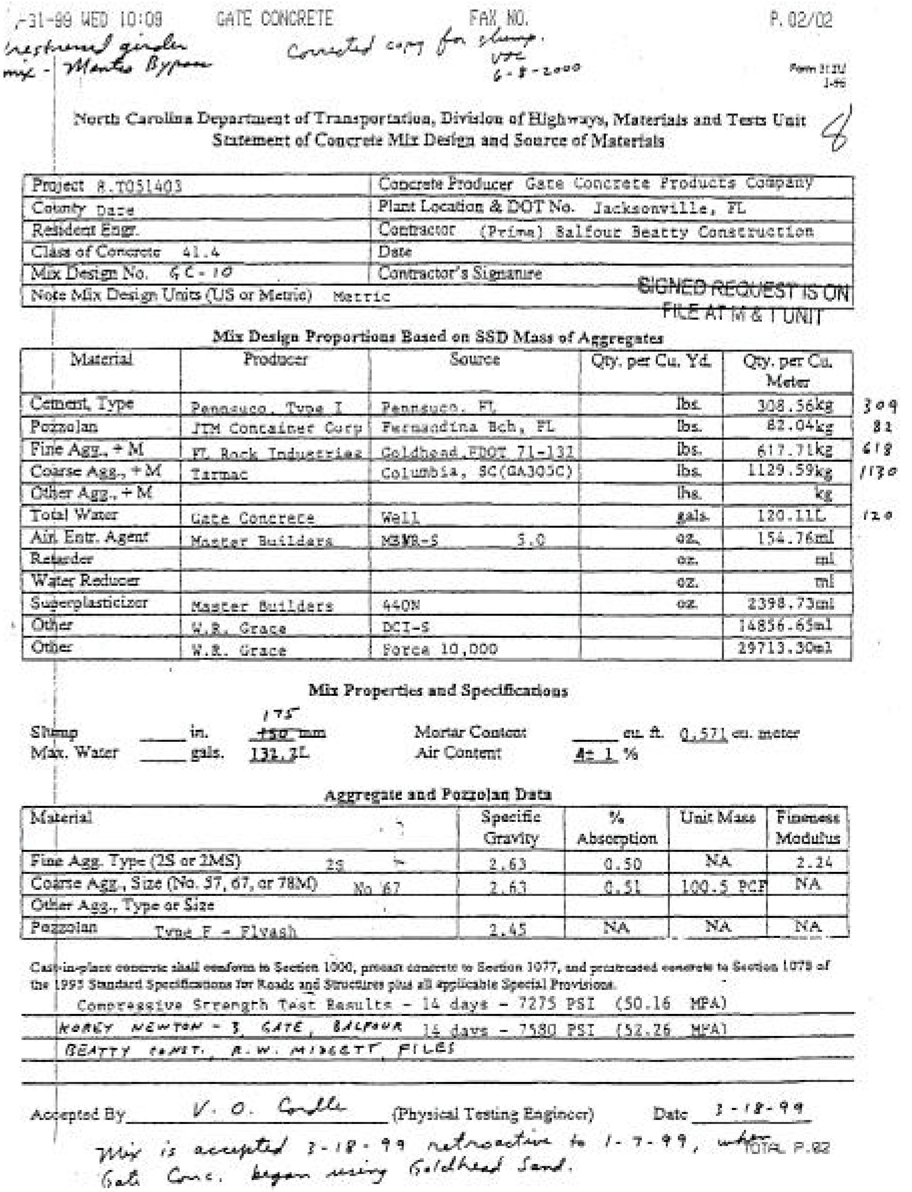
Accepted By V. O. Coble (Physical Testing Engineer) Date 9-14-99

KOREY NEWTON - 3, GATE CONC., BALFOUR BEATTY, R. W. MIDGETT, FILES TOTAL P.02

Figure A-7. Mix design 7

Table A-7. Mix results for mix design 7

Mix #	Component	Source	Quantity		Test Results			Notes	Notes	
7A	Cement	Pennsuco Cement	27.8	lb	first slump	0		AE 90 was used in place of MBVR-S, which has been discontinued. The substitution was made on the recommendation of Vincent Washington. We added 1.5 lb of water because the mix was way too dry to achieve the desired slump. 1.5 lb of water was within the "max water" limit on the mix sheet.	Date Poured	17-Jan
	Fine Aggregate	Florida Rock Industries, Grandin	39.1	lb	first air	4	chase		Batch Size	1
	Coarse Aggregate	Tarmac, Columbia	75.5	lb	second slump	8.75			Mix Type	Prestressed girder
	Water	Tap	7.2	lb	second air	4	chase			mix, Manteo Bypass
	AE 90	Master Builders	2.5	mL	third slump					
	Pozzolith 100XR	Master Builders	12.3	mL	third air	6	air pot			
	R-3000	Master Builders	41.2	mL						
	DCI-S	W.R. Grace & Company	351	mL						
7B	Cement	Pennsuco Cement	27.8	lb	first slump	5.25		We reduced the amount of water we added to .5 lb. 70 degrees.	Date Poured	17-Jan
	Fine Aggregate	Florida Rock Industries, Grandin	39.1	lb	first air	4	chase		Batch Size	1
	Coarse Aggregate	Tarmac, Columbia	75.5	lb	second slump	12"			Mix Type	Prestressed girder
	Water	Tap	7.2	lb	second air	4.5	air pot			mix, Manteo Bypass
	AE 90	Master Builders	1.5	mL	third slump					
	Pozzolith 100XR	Master Builders	12.3	mL	third air					
	R-3000	Master Builders	51.2	mL						
	DCI-S	W.R. Grace & Company	351	mL						
7C	Cement	Pennsuco Cement	27.8	lb	first slump	7			Date Poured	17-Jan
	Fine Aggregate	Florida Rock Industries, Grandin	39.1	lb	first air	3	chase		Batch Size	1
	Coarse Aggregate	Tarmac, Columbia	75.5	lb	second slump				Mix Type	Prestressed girder
	Water	Tap	7.2	lb	second air	4	air pot			mix, Manteo Bypass
	MBVR-S	Master Builders	1.5	mL	third slump					
	Pozzolith 100XR	Master Builders	12.3	mL	third air					
	R-3000	Master Builders	51.2	mL						
	DCI-S	W.R. Grace & Company	351	mL						
7D	Cement	Pennsuco Cement	62.6	lb	first slump	8		AE 90 was used in place of MBVR-S, which has been discontinued. The substitution was made on the recommendation of Vincent Washington. Mix has 1.1 lb extra water. This is within the max water limit on the mix design sheet. Adding the water seemed to be necessary to achieve the proper slump.	Date Poured	17-Jan
	Fine Aggregate	Florida Rock Industries, Grandin	87.9	lb	first air	4.5	air pot		Batch Size	2.25
	Coarse Aggregate	Tarmac, Columbia	170	lb	second slump	7.25			Mix Type	Prestressed girder
	Water	Tap	17.3	lb	second air					mix, Manteo Bypass
	AE 90	Master Builders	3.4	mL	third slump					
	Pozzolith 100XR	Master Builders	27.7	mL	third air					
	R-3000	Master Builders	115	mL						
	DCI-S	W.R. Grace & Company	789	mL						



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prestressed girder mix - Manteo Bypass

Corrected copy for slump. VOC 6-8-2000

8

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Project 8.T051403	Concrete Producer Gate Concrete Products Company
County Dade	Plant Location & DOT No. Jacksonville, FL
Resident Engr.	Contractor (Prime) Balfour Beatty Construction
Class of Concrete 41.4	Date
Mix Design No. GC-10	Contractor's Signature
Note Mix Design Units (US or Metric) Metric	

SIGNED REQUEST IS ON FILE AT M & T UNIT

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yd.	Qty. per Cu. Meter	
Cement, Type	Pennsuco, Type I	Pennsuco, FL	lbs.	308.56kg	309
Pozzolan	JTM Container Corp	Fernandina Bch, FL	lbs.	82.04kg	82
Fine Agg., + M	FL Rock Industries	Goldhead,FDOT 71-131	lbs.	617.71kg	618
Coarse Agg., + M	Tarmac	Columbia, SC(GA303C)	lbs.	1129.59kg	1130
Other Agg., + M			lbs.	kg	
Total Water	Gate Concrete	Well	gals.	120.11L	120
Air Entr. Agent	Master Builders	MBVR-S 5.0	oz.	154.76ml	
Retarder			oz.	ml	
Water Reducer			oz.	ml	
Superplasticizer	Master Builders	440N	oz.	2398.73ml	
Other	W.R. Grace	DCI-S		14856.65ml	
Other	W.R. Grace	Force 10,000		29713.30ml	

Mix Properties and Specifications

Slump ____ in. ~~150~~ 175 mm Mortar Content ____ cu. ft. 0.571 cu. meter

Max. Water ____ gals. 132.3L Air Content 4± 1 %

Aggregate and Pozzolan Data

Material		Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Agg. Type (2S or 2MS)	2S	2.63	0.50	NA	2.24
Coarse Agg., Size (No. 57, 67, or 78M)	No 67	2.63	0.51	100.5 PCF	NA
Other Agg., Type or Size					
Pozzolan	Type F - Flyash	2.45	NA	NA	NA

Cast-in-place concrete shall conform to Section 1000, precast concrete to Section 1077, and prestressed concrete to Section 1078 of the 1995 Standard Specifications for Roads and Structures plus all applicable Special Provisions.

Compressive Strength Test Results - 14 days - 7275 PSI (50.16 MPA)

KOREY NEWTON - 3, GATE, BALFOUR 14 days - 7580 PSI (52.26 MPA)

BEATTY CONST., R.W. MIDGETT, FILES

Accepted By V. O. (Physical Testing Engineer) Date 3-18-99

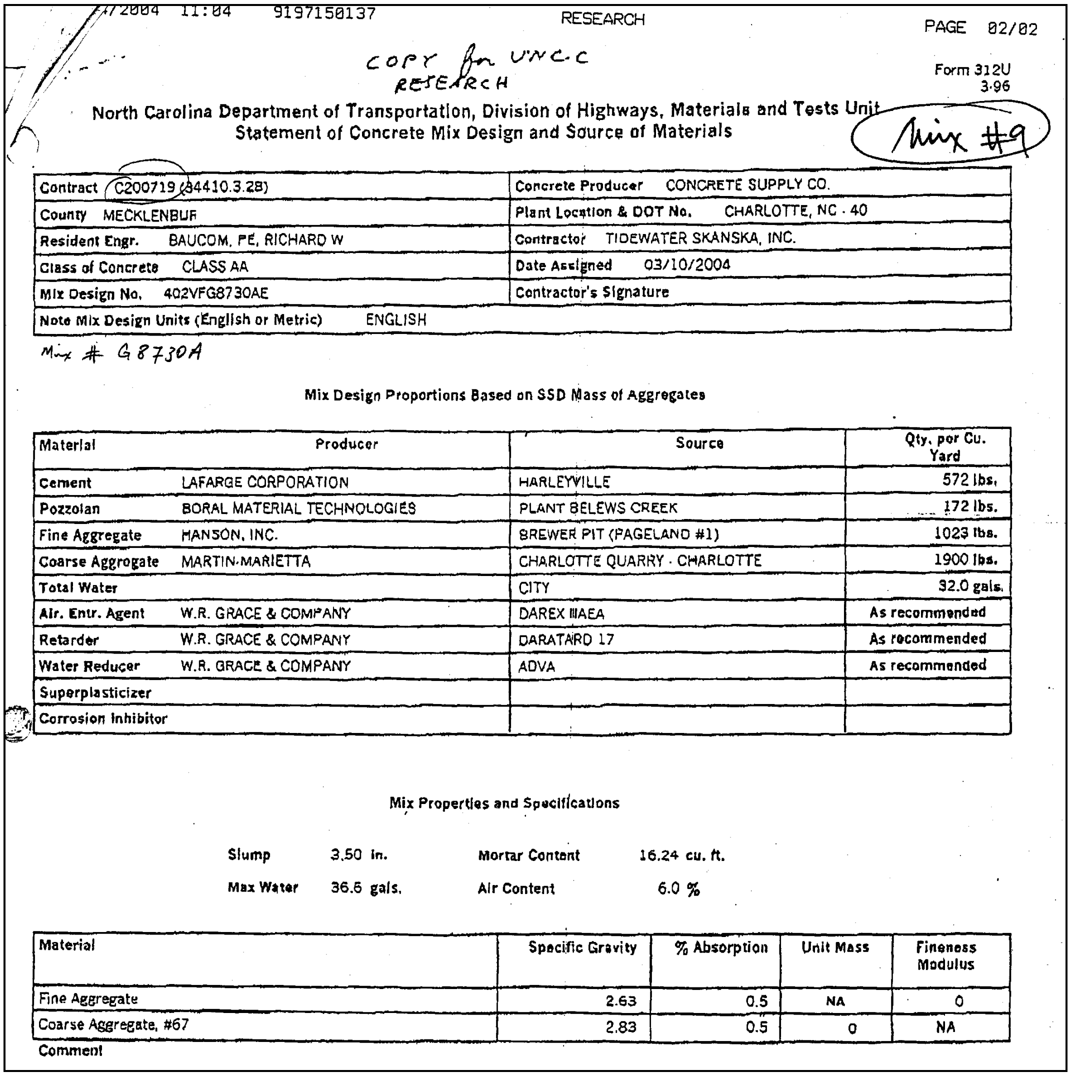
Mix is accepted 3-18-99 retroactive to 1-7-99, when Gate Conc. began using Goldhead Sand.

TOTAL P.02

Figure A-8. Mix design 8

Table A-8. Mix results for mix design 8

8A	Cement	Pennsuco Cement	23.1	lb	first slump	1.25		The force 10,000 slurry was made by mixing 6lb of microsilica with 5.6 lb of water. 12 mL of R-1000 and 5 mL of AE 90 was added mid mix and the second air and slump tests were done. An additional 12 mL of R-1000 was added and the third slump and air was measured. It was discovered that this mix should have included 9.2 lb of water	Date Poured	6-Feb
	Pozzolan	JTM Container Corp	6.1	lb	first air	2	chase		Batch Size	1.2
	Fine Aggregate	Florida Rock Industries, Goldhead	46.3	lb	second slump	3			Mix Type	Prestressed girder mix, Manteo Bypass
	Coarse Aggregate	Tarmac, Columbia	84.7	lb	second air	4	chase			
	Water	Tap	7.7	lb	third slump	3				
	AE 90	Master Builders	5.2	mL	third air	4	air pot			
	R1000	Master Builders	68	mL						
	DCI-S	W.R. Grace & Company	505	mL						
	Force 10,000	W.R. Grace & Company	1010	mL						
8B	Cement	Pennsuco Cement	19.2	lb	first slump	6		added 10 mL of R-1000 after 2nd slump test	Date Poured	6-Feb
	Pozzolan	JTM Container Corp	5.1	lb	first air				Batch Size	1
	Fine Aggregate	Florida Rock Industries, Goldhead	38.6	lb	second slump	4			Mix Type	Prestressed girder mix, Manteo Bypass
	Coarse Aggregate	Tarmac, Columbia	70.5	lb	second air					
	Water	Tap	7.7	lb	third slump	6				
	AE 90	Master Builders	4	mL	third air	5	air pot			
	R1000	Master Builders	57	mL						
	DCI-S	W.R. Grace & Company	421	mL						
	Force 10,000	W.R. Grace & Company	842	mL						
8C	Cement	Pennsuco Cement	50.9	lb	first slump	7		Air meter read 8% on second air test. We believe that the needle was sticking because it immediately settled to 6% when tapped. 6% is probably the more reliable result. Mix temperature was 64 F. 1 gallon of Force 10,000 slurry was made by combining 6 lb of microsilica with 5.6 lb water. Bill Brooks of W.R. Grace provided these proportions.	Date Poured	6-Feb
	Pozzolan	JTM Container Corp	13.6	lb	first air	7.5			Batch Size	2.65
	Fine Aggregate	Florida Rock Industries, Goldhead	102.3	lb	second slump				Mix Type	Prestressed girder mix, Manteo Bypass
	Coarse Aggregate	Tarmac, Columbia	186.8	lb	second air	6				
	Water	Tap	20.4	lb	third slump					
	AE 90	Master Builders	21	mL	third air					
	R-1000	Master Builders	178	mL						
	DCI-S	W.R. Grace & Company	1338	mL						
	Force 10,000	W.R. Grace & Company	2231	mL						



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Form 312U
3-96

North Carolina Department of Transportation, Division of Highways, Materials and Tests Unit
Statement of Concrete Mix Design and Source of Materials

Mix #9

Contract C200719 (34410.3.28)	**Concrete Producer** CONCRETE SUPPLY CO.
County MECKLENBUF	**Plant Location & DOT No.** CHARLOTTE, NC - 40
Resident Engr. BAUCOM, PE, RICHARD W	**Contractor** TIDEWATER SKANSKA, INC.
Class of Concrete CLASS AA	**Date Assigned** 03/10/2004
Mix Design No. 402VFG8730AE	**Contractor's Signature**
Note Mix Design Units (English or Metric) ENGLISH	

Mix # G8730A

Mix Design Proportions Based on SSD Mass of Aggregates

Material	Producer	Source	Qty. per Cu. Yard
Cement	LAFARGE CORPORATION	HARLEYVILLE	572 lbs.
Pozzolan	BORAL MATERIAL TECHNOLOGIES	PLANT BELEWS CREEK	172 lbs.
Fine Aggregate	HANSON, INC.	BREWER PIT (PAGELAND #1)	1023 lbs.
Coarse Aggregate	MARTIN-MARIETTA	CHARLOTTE QUARRY - CHARLOTTE	1900 lbs.
Total Water		CITY	32.0 gals.
Air. Entr. Agent	W.R. GRACE & COMPANY	DAREX IIAEA	As recommended
Retarder	W.R. GRACE & COMPANY	DARATARD 17	As recommended
Water Reducer	W.R. GRACE & COMPANY	ADVA	As recommended
Superplasticizer			
Corrosion Inhibitor			

Mix Properties and Specifications

Slump	3.50 in.	Mortar Content	16.24 cu. ft.
Max Water	36.6 gals.	Air Content	6.0 %

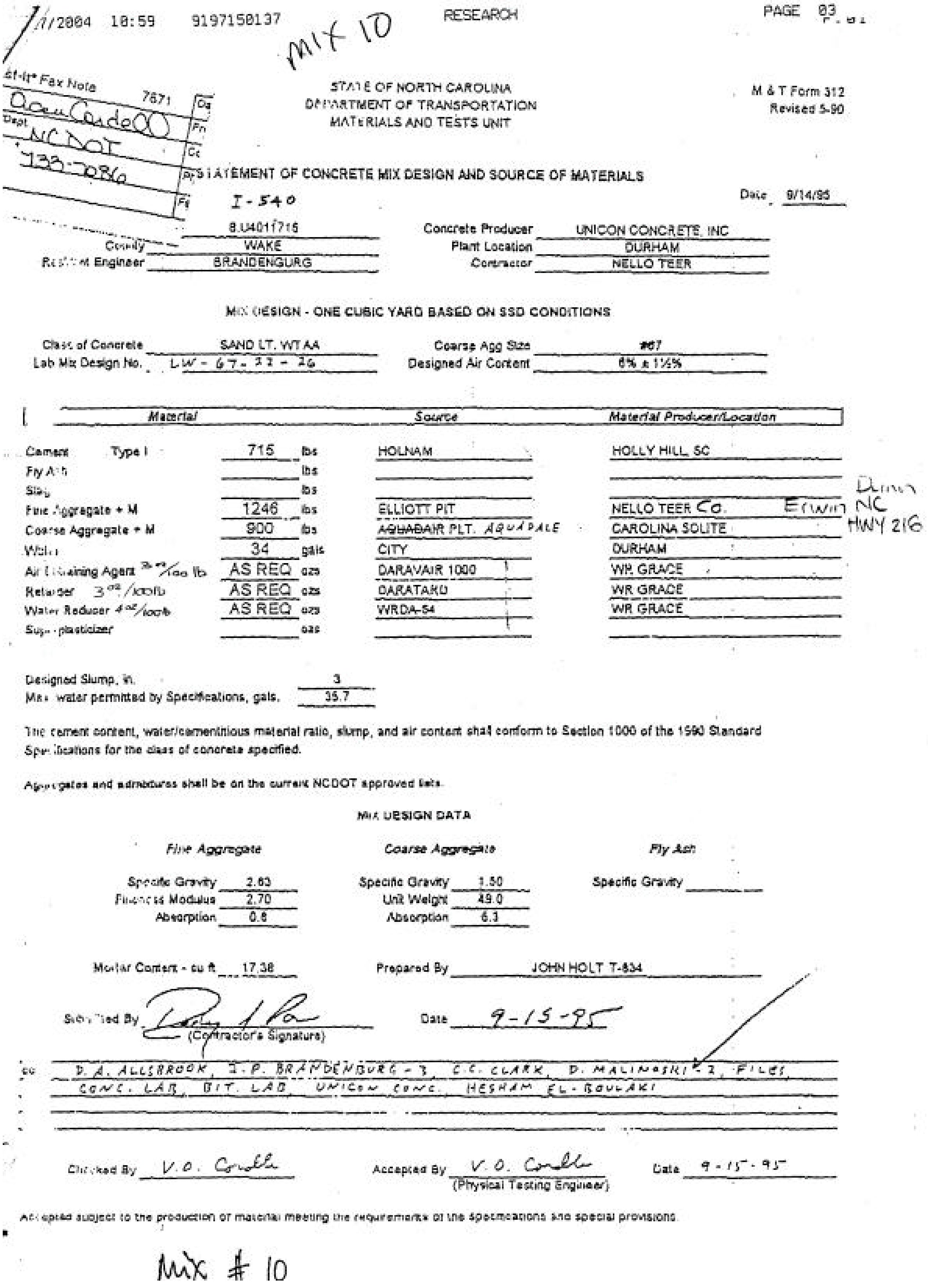
Material	Specific Gravity	% Absorption	Unit Mass	Fineness Modulus
Fine Aggregate	2.63	0.5	NA	0
Coarse Aggregate, #67	2.83	0.5	0	NA

Comment

Figure A-9. Mix design 9

Table A-9. Mix results for mix design 9

Mix #	Component	Source	Quantity	Test Results		Notes	Notes	
9	Cement	Lafarge, Harleyville	572 lb	Slump	2.5	This mix was acquired from a Concrete Supply bridge deck pour on I-485. The quantities listed are per cubic yard and the slump, air, and temperature was measured by the state personnel on site.	Date Poured	6-May
	Pozzolan	Boral Material Technologies	172 lb	Air	4.5		Batch Size	
	Fine Aggregate	Hanson, Inc	1023 lb	Temperature	68		Mix Type	Bridge deck, I-485
	Coarse Aggregate	Martin Marietta	1900 lb					
	Water	Tap	266.9 lb					
	Darex IIAEA	W.R. Grace & Company	414 mL					
	Daratard 17	W.R. Grace & Company	878 mL					



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STATE OF NORTH CAROLINA
DEPARTMENT OF TRANSPORTATION
MATERIALS AND TESTS UNIT

M & T Form 312
Revised 5-90

STATEMENT OF CONCRETE MIX DESIGN AND SOURCE OF MATERIALS

I-540

Date 9/14/95

8.U401I715

County	WAKE	Concrete Producer	UNICON CONCRETE, INC
Resident Engineer	BRANDENGURG	Plant Location	DURHAM
		Contractor	NELLO TEER

MIX DESIGN - ONE CUBIC YARD BASED ON SSD CONDITIONS

Class of Concrete	SAND LT. WT AA	Coarse Agg Size	#67
Lab Mix Design No.	LW-67-33-26	Designed Air Content	6% ± 1½%

Material			Source	Material Producer/Location
Cement Type I	715	lbs	HOLNAM	HOLLY HILL, SC
Fly Ash		lbs		
Slag		lbs		
Fine Aggregate + M	1246	lbs	ELLIOTT PIT	NELLO TEER Co. Erwin Dunn NC HWY 216
Coarse Aggregate + M	900	lbs	~~AQUADAIR~~ PLT. AQUADALE	CAROLINA SOLITE
Water	34	gals	CITY	DURHAM
Air Entraining Agent ¾ oz/100 lb	AS REQ	ozs	DARAVAIR 1000	WR GRACE
Retarder 3 oz/100lb	AS REQ	ozs	DARATARD	WR GRACE
Water Reducer 4 oz/100lb	AS REQ	ozs	WRDA-54	WR GRACE
Super plasticizer		ozs		

Designed Slump, in. 3
Max water permitted by Specifications, gals. 35.7

The cement content, water/cementitious material ratio, slump, and air content shall conform to Section 1000 of the 1990 Standard Specifications for the class of concrete specified.

Aggregates and admixtures shall be on the current NCDOT approved lists.

MIX DESIGN DATA

Fine Aggregate		Coarse Aggregate		Fly Ash	
Specific Gravity	2.63	Specific Gravity	1.50	Specific Gravity	
Fineness Modulus	2.70	Unit Weight	49.0		
Absorption	0.8	Absorption	6.3		

Mortar Content - cu ft 17.38
Prepared By JOHN HOLT T-834

Submitted By (Contractor's Signature)
Date 9-15-95

cc D. A. ALLSBROOK, J. P. BRANDENBURG - 3, C. C. CLARK, D. MALINOSKI - 2, FILES,
CONC. LAB, BIT. LAB, UNICON CONC., HESHAM EL-BOULAKI

Checked By V. O. Cordle
Accepted By V. O. Cordle (Physical Testing Engineer)
Date 9-15-95

Accepted subject to the production of material meeting the requirements of the specifications and special provisions.

Mix # 10

Figure A-10. Mix design 10

Table A-10. Mix results for mix design 10

Mix #	Component	Source	Quantity	Test Results		Notes	Notes	
10A	Cement	Holcium	39.7 lb	first slump	1.5	Added 15 mL of WRDA-35 initially and added 10 mL after the first slump test. It was discovered after the mixing that there should have been 16.4 lb of water instead of 16.2 lb.	Date Poured	7-May
	Fine Aggregate	Erwin, NC	68.6 lb	first air	5.25		Batch Size	1.5
	Coarse Aggregate	Aquadale Solite	50 lb	second slump	2		Mix Type	Lightweight bridge deck
	Water	Tap	16.17 lb	second air				
	Daravair 1000	W.R. Grace & Company	3 mL	Temperature	78.8			
	Daratard 17	W.R. Grace & Company	15 mL					
	WRDA-35	W.R. Grace & Company	25 mL					
10B	Cement	Holcium	26.5 lb	first slump	2		Date Poured	7-May
	Fine Aggregate	Erwin, NC	45.7 lb	first air	5.75		Batch Size	1
	Coarse Aggregate	Aquadale Solite	33.3 lb	second slump	2.5		Mix Type	Lightweight bridge deck
	Water	Tap	10.86 lb	second air				
	Daravair 1000	W.R. Grace & Company	3 mL	Temperature	85.1			
	Daratard 17	W.R. Grace & Company	15 mL					
	WRDA-35	W.R. Grace & Company	25 mL					
10C	Cement	Holcium	26.5 lb	first slump	2	The first air pot leaked so the air test was performed a second time.	Date Poured	7-May
	Fine Aggregate	Erwin, NC	45.7 lb	first air	4		Batch Size	1
	Coarse Aggregate	Aquadale Solite	33.3 lb	second slump	1.5		Mix Type	Lightweight bridge deck
	Water	Tap	10.86 lb	second air	5			
	Daravair 1000	W.R. Grace & Company	3 mL	Temperature	82.4			
	Daratard 17	W.R. Grace & Company	15 mL					
	WRDA-35	W.R. Grace & Company	30 mL					
10D	Cement	Holcium	66.2 lb	first slump	2.5	Initially there was 62.5 mL of WRDA in the mixture. After the first slump test 8 mL were added. The air went high on this test and that was possibly due to the extremely high temperature of the concrete mixture.	Date Poured	7-May
	Fine Aggregate	Erwin, NC	114.3 lb	first air	8.5		Batch Size	2.5
	Coarse Aggregate	Aquadale Solite	83.3 lb	second slump	2.5		Mix Type	Lightweight bridge deck
	Water	Tap	27.2 lb	second air				
	Daravair 1000	W.R. Grace & Company	12.5 mL	Temperature	89.6			
	Daratard 17	W.R. Grace & Company	37.5 mL					
	WRDA-35	W.R. Grace & Company	70.5 mL					