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# Effects of Wet Mat Curing Time and Earlier Loading on Long-Term Durability of Bridge Decks: Rapid Chloride Permeability Tests and Ponding Test

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

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This research report is one of several published from a six-year study sponsored by the Texas Department of Transportation (TxDOT) that investigated the impact of wet-mat curing duration on bridge deck concrete durability. This report presents findings related to chloride permeability of several widely used TxDOT bridge deck concrete mixes. The Rapid Chloride Permeability Test (RCPT) and the Ponding Test (AASHTO T259) were selected and seven mix designs were tested; four were evaluated using cores taken from field-cast model bridge decks and the other three from laboratory-cast specimens. Mix designs were selected based on their extent of use around the State, with differences in terms of type of cement, coarse aggregate type, supplementary cementitious material (SCM) type and SCM content. Wet-mat curing durations of 0, 2, 4, 8, 10 and 14 days were evaluated for each mix. The chloride permeability of each mix was found to be sensitive to the wet-mat curing duration and each mix appeared to provide increases in resistance to permeability up to 4 or 8 days depending on the mix, beyond which the marginal benefits appeared to diminish. Both tests, the RCPT in particular, showed sensitivity to mixes that contained SCMs. Mixes that contained siliceous gravel coarse aggregate and Type I cement showed some of the highest permeability values among all mixes tested. They displayed moderate water soluble integral chloride (ponding) and electric charge (RCPT) values. It was also found that permeability of concrete decreased as percentage of fly ash (either class F or class C) increased from around 20% to around 30%. For the seven mixes evaluated, the ponding test results showed that steel reinforcement can be put at a depth 1.25" when compared with threshold chloride contents commonly used in the literature. A comparison of results obtained from the two test methods showed a correlation between their results for some mixes. This may indicate that RCPT does not correlate well with actual long-term chloride penetration for all mixes. Further research is needed to either develop a new test or evaluate an alternative test method that can provide a quick direct measurement of the chloride ion concentration rather than an indirect process where electrical conductivity is used.

17. Key Words: concrete, bridge, deck, durability, curing, wet mat, vehicle loading, early loading

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# EFFECTS OF WET MAT CURING TIME AND EARLIER LOADING ON LONG-TERM DURABILITY OF BRIDGE DECKS: RAPID CHLORIDE PERMEABILITY TESTS AND PONDING TESTS

by

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

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# CHAPTER I
# INTRODUCTION

In a combined effort between TxDOT and Texas Tech University (TTU), researchers at TTU began a process that potentially could lead to the development of new guidelines as to when bridge decks can be open to (a) construction traffic and (b) full traffic without compromising concrete durability. To achieve this objective, various mix designs were subjected to different curing durations to observe which, if any, had an effect on the durability of concrete. Wet-mat curing test durations were for 2, 4, 8, 10, and 14 days from the time of pour. Also, a “no moist curing” option is considered; it is hereafter referred to as 0 day cure.

Seven concrete mixture designs widely used around the State of Texas were selected to evaluate their durability using several criteria.  Cast specimens and field cores were subjected to a number of tests to evaluate the durability of concrete; Rapid Chloride Permeability Test (RCPT,) chloride ponding, compressive strength, freeze-thaw resistance, petro graphic analysis, tensile strength, shrinkage, and fracture toughness. This report focuses on the RCPT and ponding tests.

Each mix was tested to assess chloride permeability using the Rapid Chloride Permeability Test (RCPT) and the Chloride Ponding Test.  Of these seven mix designs, four were tested using specimens from field sites where bridge decks were constructed.  These were mixture designs from El Paso, Fort Worth, San Antonio and Houston districts.  Two mixture designs had specimens cast at Texas Tech University laboratories using raw materials brought in from two additional regions (Pharr and Atlanta districts) and one mix design (Lubbock) had specimens cast in the field using concrete from an actual bridge deck construction site.  At each field site, a relatively small-scale, full-size bridge deck was constructed.  In addition, numerous concrete specimens were cast alongside the bridge deck. The field bridge decks were constructed at the same districts where the selected mix designs were being used using actual contractor crews wherever possible.  This way, the durability of the mix designs could be assessed under their local conditions.

The RCPT test specimens were 4-inch diameter cores taken from the four model bridge decks in El Paso, Fort Worth, San Antonio and Houston districts. Laboratory cast 4-inch diameter cylindrical concrete specimens were used for Pharr and Atlanta mixtures, and the Lubbock RCPT specimens were 4-inch diameter field-cast specimens instead of cores because concrete was used from a real bridge deck project. All ponding tests were conducted on cast concrete specimens.

## 1.1 Mixture Designs

The mixture designs were selected from El Paso, Fort Worth, San Antonio, Houston, Pharr, Atlanta, and Lubbock districts. A predominantly-used local mixture design was used for each of the model bridge decks constructed in El Paso, Fort Worth, San Antonio and Houston districts. Ground granulated blast furnace slag (replacing 50% cement) was used at the El Paso test site. The sites in Fort Worth and Atlanta used class F fly ash replacing 22% and 21% of the Portland cement, respectively. In the San Antonio and Houston bridge decks, class C fly ash was used to replace about 20% and 27% cement respectively, and the originally selected Lubbock mixture design contained no mineral admixture. Results from RCPT performed on this mixture showed permeability values that were extremely high. For this reason, a second Lubbock mix design containing 31% fly ash as cement replacement was used (hereafter referred to as New Lubbock). The information on all district mixture designs is summarized in Tables 1.1 and 1.2 on the following pages.

**Table 1.1** Mix Designs Used in Bridge Decks in Different Districts of Texas

| **Parameter** | **Districts** | | | | | | |
|---|---|---|---|---|---|---|---|
| | San Antonio | El Paso | Pharr | Fort Worth | New Lubbock | Atlanta | Houston |
| Cement Type | I/II | I/II | I | I/II | I/II | I | I/II |
| Mineral Admixture | Fly ash C 20% | Slag 50% | None | Fly ash F 22% | Fly ash F 31% | Fly ash F 21% | Fly ash C 27% |
| Coarse Aggregate | LS | LS | Sil | LS | LS | Sil | LS |
| W/C | 0.43 | 0.42 | 0.41 | 0.43 | 0.45 | 0.46 | 0.46 |
| Air | 6% | 5% | 6% | 6% | 6% | 6% | 5% |

LS = Limestone

Sil = Siliceous gravel

**Table 1.2** Summary of Volumetric Concrete Mix Design for Lubbock, El Paso, Fort Worth and San Antonio Sites

| | Lubbock | El Paso | Fort Worth | San Antonio |
|---|---|---|---|---|
| Coarse Aggregate | 11.66 | 10.15 | 10.72 | 10.86 |
| Fine Aggregate | 6.79 | 7.55 | 7.53 | 6.56 |
| Cement | 3.03 | 1.55 | 2.36 | 2.42 |
| Water | 4.17 | 4.13 | 3.53 | 4.17 |
| Fly Ash | 0 | 0 | 0.81 | 0.75 |
| GGBS | 0 | 1.71 | 0 | 0 |
| Entrained air | 1.35 | 1.89 | 2.05 | 2.23 |
| Yield | 27.00 | 26.99 | 27.00 | 27.00 |
| Cement type | I/II | I/II | I/II | I/II |
| Water/cement ratio by mass | 0.44 | 0.42 | 0.36 | 0.44 |

Note: (1) Unit: $ft^3/yd^3$ concrete.

(2) For the volume calculations, the following specific gravities were assumed for the Fort Worth and San Antonio concrete mix design. Coarse aggregate: 2.689, Fine aggregate: 2.670, Cement: 3.10, Fly Ash: 2.6.

## 1.2 Test Specimens

Test sites were designed and cast to realistically model actual TxDOT bridge design and construction procedures as closely as possible. A TxDOT Class "S" concrete mixture design was used for each concrete deck mixture. Concrete was transported using mobile mixer trucks from a TxDOT approved ready-mix concrete plant.

### 1.2.1 Rapid Chloride Permeability Test (RCPT).

The test slab sites, selected based on climate and local construction conditions such as aggregate type and mixture proportioning, were used to cast model bridge decks using local materials and weather conditions. Test bridge decks constructed at each site consisted of six deck slabs; each cured for different duration; 0, 2, 4, 8, 10 and 14 curing days, respectively (Figure 1.1). After the casting was completed for each of the sites located in Lubbock, El Paso, Fort Worth and San Antonio, field cast test specimens and cores from the bridge deck slab were collected based on the schedule of laboratory tests. Lab tests included the rapid chloride permeability test (AASHTO T277), as well as other tests including petrographic examination of concrete cores, compressive strength test, abrasion test, flexural strength test and concrete fracture test.

**Figure 1.1** Coring of Bridge Deck Concrete for Permeability Test - El Paso Site

### 1.2.2 Ponding.

Square 12 inch ×12 inch specimens were cast as shown in Figure 1.2. Since the primary objective of this research project involves evaluation of the effects of different curing durations on the durability of concrete, concrete specimens were removed from water according to predetermined durations. As mentioned above, the curing periods studied are 0, 2, 4, 8, 10, and 14 days. Curing periods were measured and terminated based on actual calendar days irrespective of any extreme temperature fluctuations that might have occurred.

**Figure 1.2** Casting of the 12”×12” Specimens

### 1.3 Specimen Identification

For all the specimens of the test sites, specific codes were assigned as given in Fig. 1.3,

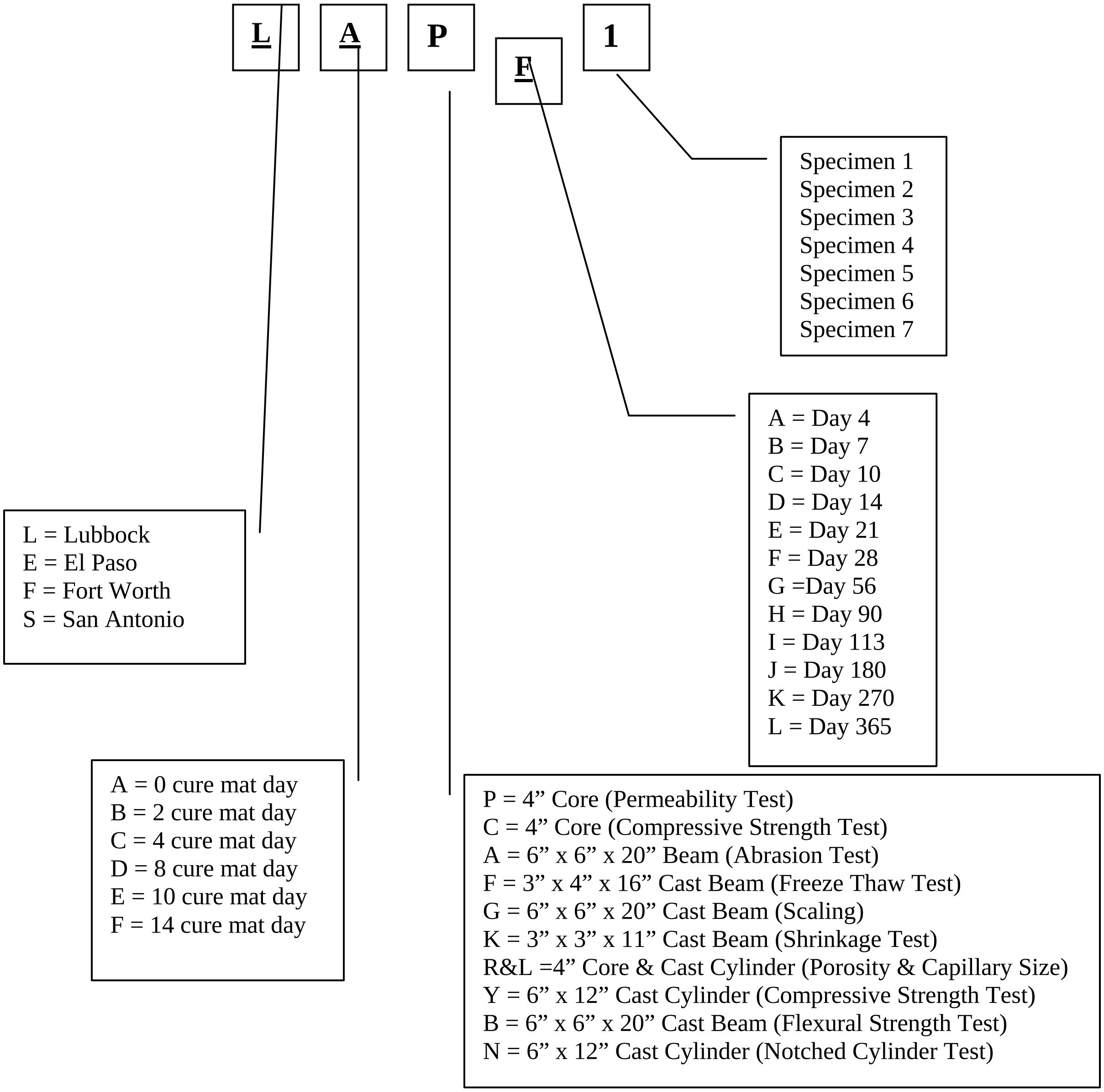


**Figure 1.3** Coding System Used for Specimen Identification

# CHAPTER II
# TEST METHODS

## 2.1 Rapid Chloride Permeability Test (RCPT)

This test, which has the dual designation of ASTM C 1202-97 and AASHTO T 277 is titled *Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration.* It is widely used as it covers the laboratory evaluation of the electrical conductance of a concrete sample to provide a rapid indication of its resistance to chloride ion penetration. In 1982, researchers presented the RCPT laboratory method to AASHTO for consideration as a standard test method and AASHTO accepted the test method. In late 1991, this method was also adopted by ASTM (Whiting and Mitchell 1992).

### 2.1.1 Preparation and Procedure:

#### *2.1.1.1 Sample Preparation*

1. After removing cylindrical permeability cores from the bridge decks the cores are kept moist until cut into 2-inch thick discs (i.e. each disk is 4 inches in diameter and 2 inches in thickness). The 2-inch thick disk is taken from the top of the core (leaving topmost ½ inch slice) using water-cooled diamond saw.
2. Epoxy is applied to the side surface of each disk specimen. This epoxy coating is allowed to cure or dry until it is no longer sticky to the touch (at least 8-10 hours).
3. Distilled water is placed in a large container, which is connected to a vacuum pump in order to de-air the water.
4. Specimen are placed into vacuum desiccators in such a way that both end surfaces of the specimen are exposed. The desiccators are sealed using the vacuum pump; the desiccators de-air simultaneously both the specimen and the water for 3 hours.
5. Once vacuuming of the water container is complete, a sufficient amount of water is allowed to fill the desiccators so that the cores are covered with de-aerated water.
6. The vacuum pump is allowed to run for another hour to de-air the submerged specimen.

7. The pump is turned off and the specimen are left in the de-aerated water in the desiccators for 16-20 hours to make the cores fully saturated.
8. Specimens are removed from the desiccators after the specified time of soaking is complete.
9. Using RTV silicon rubber or silicon rubber caulking sealants, specimens are caulked to the Plexiglas cells. The specimens are then left for 12-24 hours.

### *2.1.1.2 Test Procedure*

1. At the conclusion of the drying period, the specimen is ready for the RCPT test. One side of the cells is filled with 3.0% NaCl solution (the side connected to the negative terminal of the 60 V power supply) and the other side is filled with 0.3N (Normal) NaOH solution (the side connected to the positive terminal of the power supply).
2. Just after the test is started, initial current and charge readings are taken. Subsequent readings are taken every 30 minutes. Temperature readings are taken along with the current and charge readings at each time interval.
3. During the test, should the temperature of either solution exceed 190° F (90°C), the test must be aborted to avoid damage of the cells and/ or boiling of either solution.
4. The RCPT test is terminated after 360 minutes of continuously applied voltage.

RCPT data reflects the electrical resistivity of the concrete being tested rather than the resistance of the concrete to chloride penetration. Therefore the test is an indirect way of measuring concrete permeability. There have been a number of criticisms of this method, including:

a. Current passed is related to all ions in the concrete pore solution and not just the chloride ions.
b. Measurements are made prior to the concrete reaching steady state ion state migration.
c. The high voltage applied leads to an increase in temperature, especially for high permeability concretes, which increases the charge passed even more.

Each of these criticisms is addressed in the text to follow. During the RCPT, concrete generates more heat with high permeability, as temperature rise is proportional to the product of the current and voltage. For example, the higher the permeability of a given concrete mix, the greater the resulting current at a given voltage. Thus, the greater the heat energy produced. This heating leads to a further increase in the charge passed-over and above the charge if the temperature remained constant. Thus, poor quality concrete may look even worse using RCPT than it would actually behave otherwise (Hooton et al., 1997).

Another concern of the RCPT is that its results depend upon the relationship between concrete conductivity and chloride ion penetrability. Any conducting material embedded in the concrete specimen will influence the test result, such as the presence of reinforcing steel, conductive fibers (carbon or steel), or a highly conductive pore solution. Therefore, any such condition should be noted in the test results.

A more recent study (Walsh et al., 1998) indicates that more than 99% of the chloride ion does not pass completely through the concrete disk specimen during the RCPT, but instead remains within the concrete disk. When the test is terminated after six hours, it was shown that generation of the charge not only involves migration of ions, but also the electrolysis of water at each of the two electrodes in the cell system of RCPT. This charge due to the electrolysis of water is always present as background charge, which gives the false impression of charges due to the ions present in concrete. Among all other available short-term method, RCPT is well known and widely used. If any correlation with a long-term method can be established for the same mix design, RCPT can be used effectively as the best permeability test method.

## 2.2. Chloride Ion Penetration Test

This test has the designation AASHTO T 259 and is commonly referred to as either the “Salt Ponding Test” or “Soaking Test”. It is a long-term test to measure the chloride ion penetration into concrete. This test is often used to validate the results of the RCPT.

This test requires a minimum of 3 slabs, each having a thickness of at least 3 inches (75mm) and a surface area of 12 inches (300 mm) square. Slabs typically are moist-cured for 14 days and stored in a controller room at 50% relative humidity for 28 days. The sides of the slabs are typically sealed as shown in Fig. 2.1 on the following page, but the bottom and top face are not.

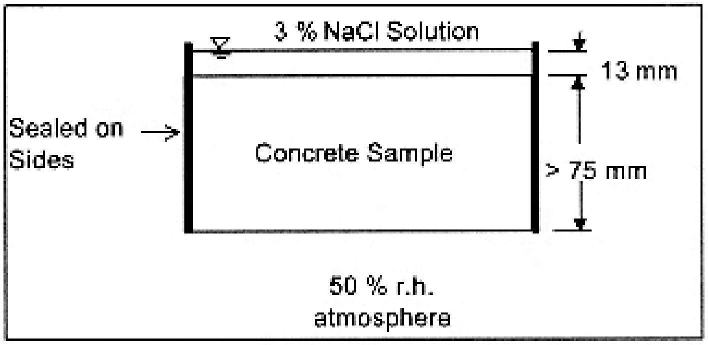


**Figure 2.1** Salt Ponding Test Setup (AASHTO T 259) (Hooton, 1997).

After the initial specimen-conditioning period, a 3% sodium chloride solution is ponded on the top surface of the slab for 90 days, while the bottom face is left exposed to the drying environment. At the end of 90 days, the slabs are removed from the drying environment and the chloride concentration of 0.5 inch-thick slices is determined. Typically, two or three measurements are taken at various depths.

In the Ponding Test, only the average chloride concentration in each 0.5-inch slice is determined, i.e., the actual variation of the chloride concentration over that 0.5 inch slice is not determined. The ponding test provides a crude one-dimensional chloride ingress profile, but this profile is not only a function of chloride diffusion. Since the specimens have been left to dry for 28 days, there is an initial sorption effect when the slabs are first exposed to the solution. Salt solution is drawn quickly into the pores of the concrete. Also the exposure of the bottom face to a 50% relative humidity environment during the test causes chlorides to be drawn into the concrete through a mechanism other than pure diffusion. There is vapor transmission from the wet front in the concrete to the drier atmosphere at the external face, causing more water to be drawn into the concrete and bringing chloride ions with it. This effect is called wicking (Hooton et al., 1997).

The ponding test overemphasizes the importance of sorption and to a lesser extent wicking. The relative amount of chloride pulled into the concrete by capillary adsorption to the amount entering by diffusion will be greater when the test is run for only 90 days, compared to the relative quantities entering during the lifetime of a structure. Also, if wicking occurs in the concrete specimen, the relative humidity gradient within it for part of the time will be less than the total set up during the test (Hooton et al., 1997).

# CHAPTER III

# RCPT TEST PROGRAM AND ANALYSIS OF RESULTS

## 3.1 Rapid Chloride Permeability Test (RCPT)

### 3.1.1 Test Program

For testing the chloride ion permeability of the concrete, core specimens were taken from model bridge deck slabs from Lubbock, El Paso, Fort Worth and San Antonio sites and the raw materials for the laboratory samples were taken from the Pharr and Atlanta sites. For each test site, at least three 4-inch diameter cores were taken from each wet mat cure day panel (0, 2, 4, 8, 10 and 14 cure day) in the test slab. A total of 18 cores of particular maturation age from a specific site were tested at a time. The core, which included reinforcement bar, was discarded for the test. These test process requires three steps; specimen preparation, conditioning and the electrical conductive test.

**Figure 3.1** RCPT Test Setup

In addition to the effect of wet-mat curing duration, the effect of the different concrete mix designs on permeability of concrete was also investigated by comparing the results from the four sites. In all cases, the analysis was focused on the variations of electrical conductivity of concrete to indicate permeability with wet-mat curing duration.

### 3.1.2 Analysis of Chloride Permeability Test Results

The experimental setup was to investigate the effect of wet-mat curing age on chloride ion permeability in the Rapid Chloride Permeability Test (RCPT), T277. Current and charge readings were taken during six-hour duration of the test at 30-minute intervals including the initial reading. By knowing the current and time history, the total charge in coulombs that passed through the specimen is computed by Simpson's integration rule. Also, temperature was monitored during the test to ensure that the charge-passed data are not affected by excessively high temperature, also referred to as the Joule effect (Detwiler, 1993). It was found that for each specimen of all the sites, there are some initial current readings. Test specimens having reinforcement bar inside were discarded.

#### *3.1.2.1 Lubbock Site*

For Lubbock site, concrete was mixed at a water-cement ratio of 0.44 with a cement-fine aggregate-coarse aggregate ratio of 1:2.24:3.85 (volumetric). The coarse aggregate used was limestone (Phelan et al., 1999). No supplementary cementing material was used in concrete of Lubbock site. Five different maturity levels of concrete were studied: 10, 14, 56, 90 and 365 days. At the start of the test, the electrodes of the reservoir cells for the final permeability test were of copper mesh. It was found that this copper mesh electrode itself corroded and gave high permeability values. At the start of Lubbock 56-day permeability test, steel electrode was used instead of this copper mesh electrode. So the tests before Lubbock 56-day permeability test were discarded.

Figure 3.2 shows the plot of current passed versus time for 56-day core specimens. It was found that current is inversely proportionate to the number of curing days. This may be due to the possibility that with an increase in curing days, concrete microstructure has been changed, thus changing the electrical conductivity of concrete. In comparison with 0-cure day concrete, 2-day cure concrete showed less current reading as well as charge passing reading. This may be due the fact that during the 0-cure day concrete permeability test run, temperature of solution reached 135°F at the end of the test.

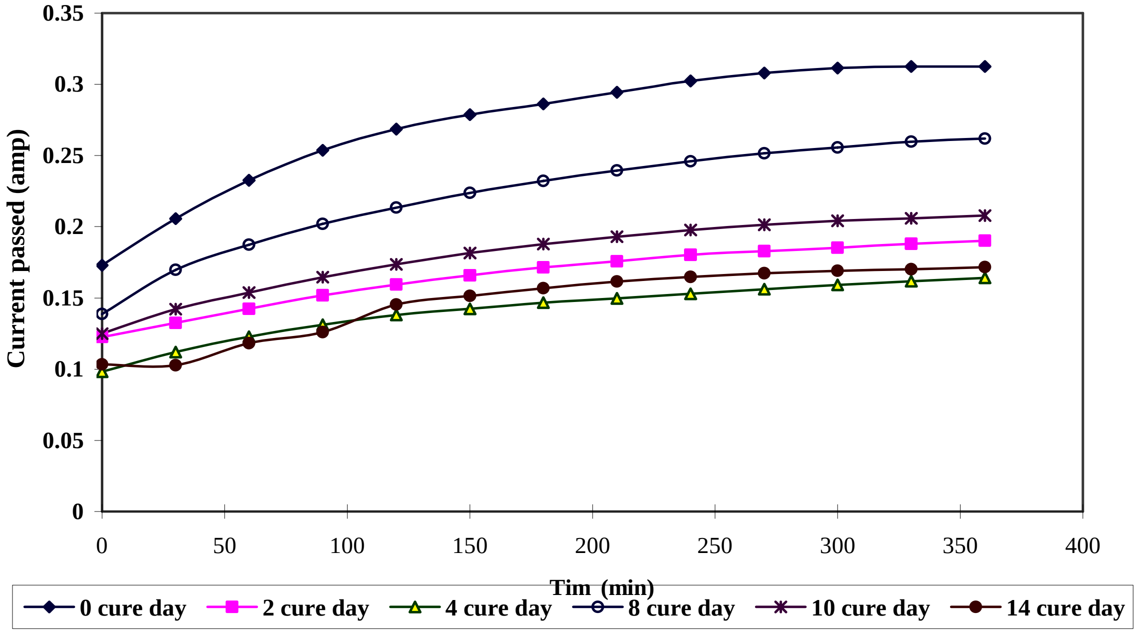


**Figure 3.2** Electrical Current Passed vs. Time for Lubbock 56-day age Specimens

In the case of 2-day wet-mat cure concrete, the temperature was 116°F. Maximum recommended temperature is 195°F.

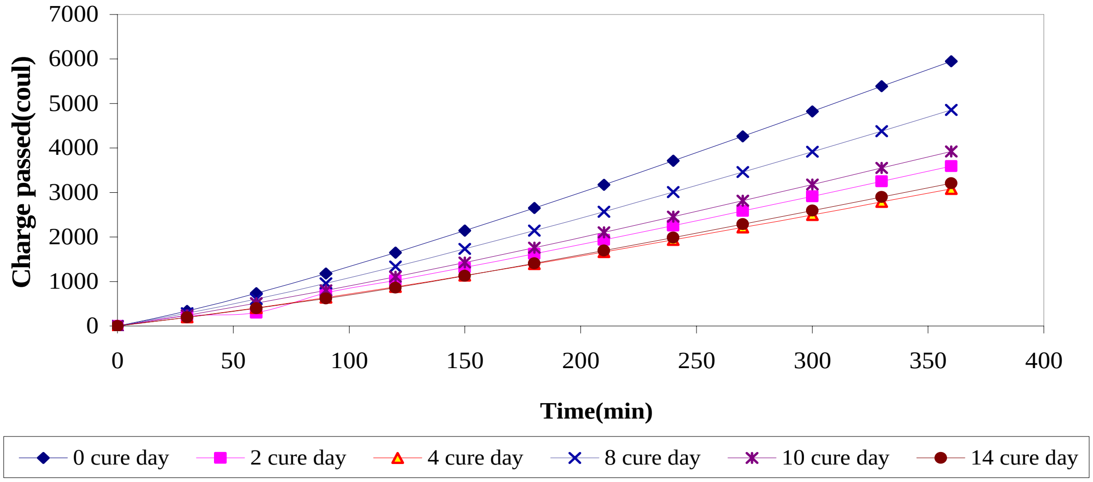


**Figure 3.3** Electrical Charge Passed vs. Time for Lubbock 56-Day Age Specimens

Fig. 3.3 shows the charges passed through Lubbock 56-day permeability cores plotted versus time elapsed during test. Table 3.1 illustrates the guidelines developed to evaluate concrete permeability based on correlations from RCPT and 90-day ponding tests.

Figures 3.4 to 3.6 show the charge passed for Lubbock specimens with ages of 56, 90 and 365 days respectively. These plots also show the minimum, maximum and average values of replicates within each curing duration. These 3 charts are then combined in Figure 3.7.

**Table 3.1** Chloride Permeability Based on Charge Passed (Whiting, 1981)

| Charge Passed (Coulombs) | Chloride Ion Penetrability |
|---|---|
| > 4000 | High |
| 2000-4000 | Moderate |
| 1000-2000 | Low |
| 100-1000 | Very Low |
| <100 | Negligible |

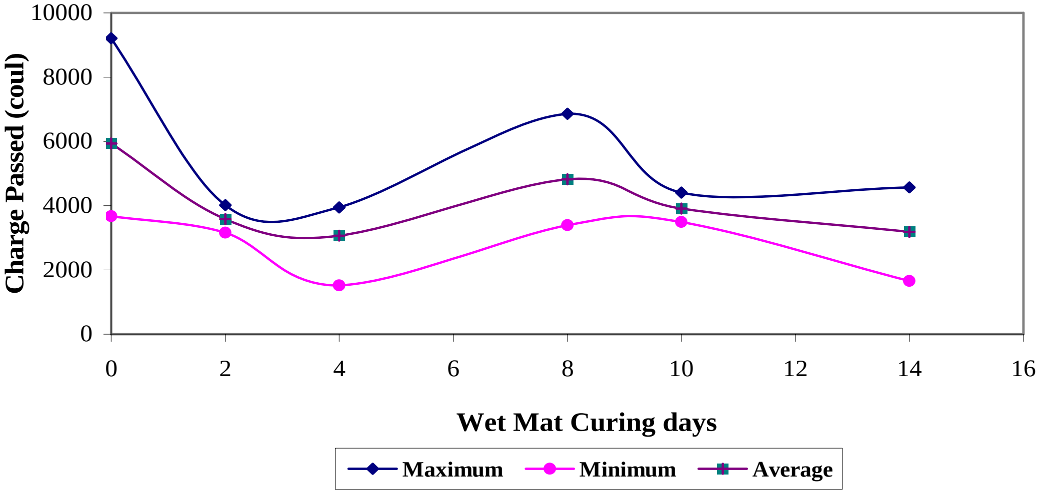


**Figure 3.4** Lubbock 56-Day Permeability for Different Wet Mat Cure Days

Table 3.2 shows the temperatures of the pore solution measured at the end of the RCPT. As shown in Fig. 3.7, the overall charges passed through concrete cores fell in the category of very high permeability.

Also, the 90-day permeability values were higher than 56-day permeability values because during the test of 90-day permeability cores, temperature of cores raised. The room temperature was 70°F. Towards the end of the test, the average temperatures of the Lubbock cores are as follows:

**Table 3.2** Temperature of Lubbock Permeability Cores (Maximum allowed 195°F)

| Lubbock Test | Temperature after 360min test run (°F) | | | | | |
|---|---|---|---|---|---|---|
| | 0-cure day concrete | 2-cure day concrete | 4-cure day concrete | 8-cure day concrete | 10-cure day concrete | 14-cure day concrete |
| 56-day | 135 | 116 | 105 | 110 | 105 | 100 |
| 90-day | 127 | 124 | 126 | 126 | 109 | 124 |
| 365-day | 128 | 115 | 103 | 102 | 100 | 99 |

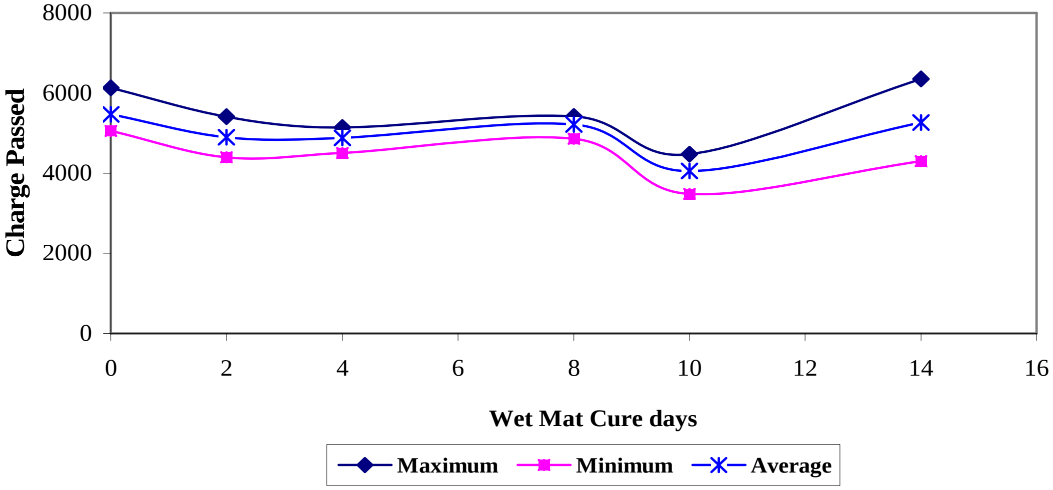


**Figure 3.5** Lubbock 90-Day Permeability Test Result

In the case of 365-day test results, it was observed that though the 0-cure day permeability was too high, the 14-cure day permeability was moderate (Fig.3.6). Results of 365-day age specimens showed lower permeability value than 90-day age specimens. Age of concrete has a significant effect on the permeability of concrete.

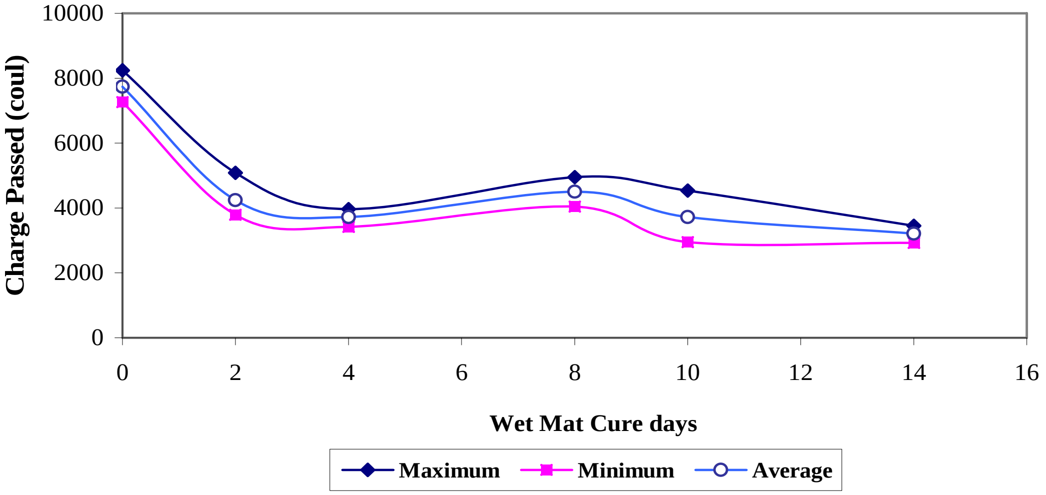


**Figure 3.6** Effect of Lubbock 365-Day Permeability Test on Charge Passed

From Table 3.2 and Fig. 3.7, it can be observed that with increase in temperature, both the current and charge of concrete increases. The phenomenon takes place because with the passage of current temperature of the solution increases. At the same time the conductivity of the pore solution of concrete is increased by the rise in temperature, which again results in current increase.

From Fig. 3.2 and Fig. 3.8, it was observed that initial currents were not much lower than the final current reading and towards the end of the test the slope of the plots became almost horizontal for almost all cure day concretes. This may be due to the fact that, initially, hydroxyl and sodium ion migrated into the specimen, increasing the conductivity of the pore water; the source of ions from the initially saturated sodium hydroxide solution was rapidly decreasing. And as hydroxyl ion is more conductive than chloride ion due to depletion of hydroxyl ion, no further significant increase in current is observed.

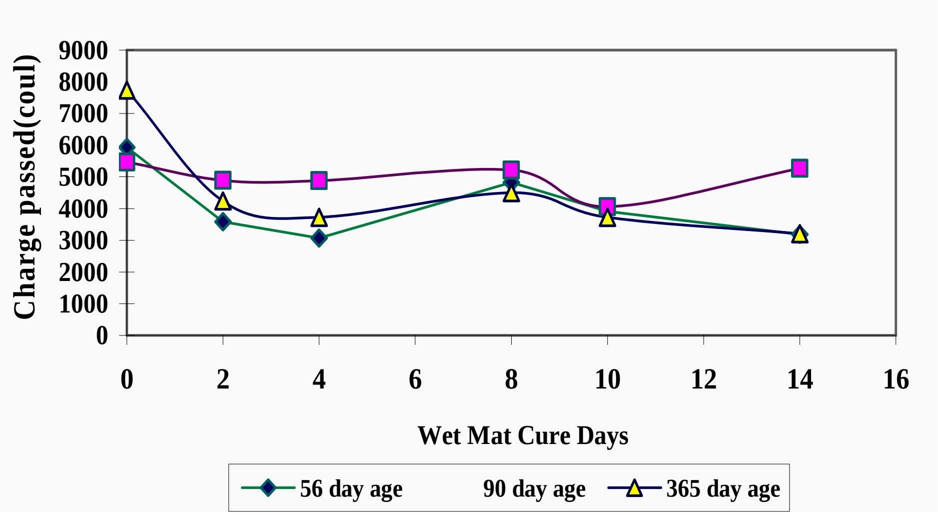


**Figure 3.7** Lubbock Overall Permeability Test Results

Fig 3.9 represents the plot of initial current versus charge passed through 14-cure day concrete. The 365-day concrete shows an approximately 37% decrease in initial current with respect to 90-day concrete. This can be explained if it is assumed that there is a greater migration of ions from concrete core at the level of impermeability obtained at 365 days than diffusion into the specimen from the sodium hydroxide solution.

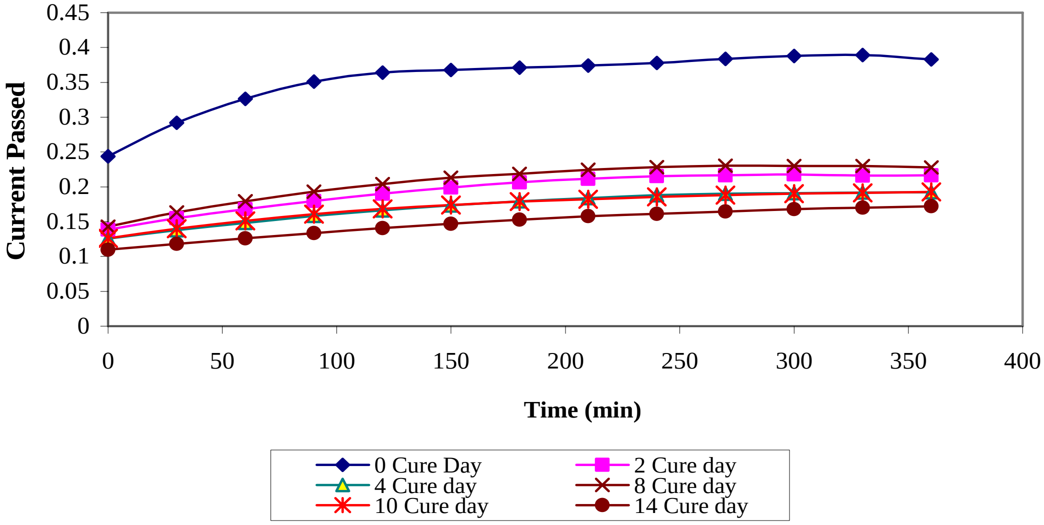


**Figure 3.8** Effect of Lubbock 365-day Permeability Test on Current Passed

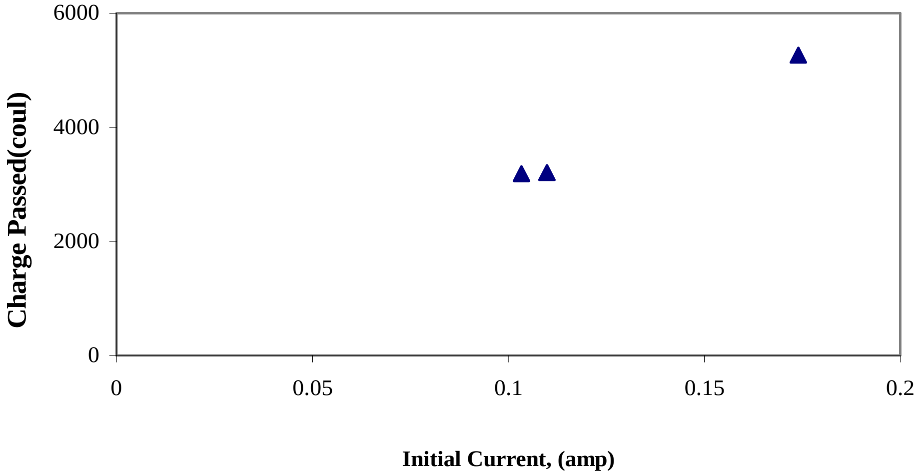


**Figure 3.9** Plot of Initial Current versus Charge Passed Through 14-Wet Mat Cure Day Concrete for Lubbock Site

### *3.1.2.2 El Paso Test Slab*

For the El Paso site, concrete was mixed at a water-cement ratio of 0.42 and 50 percent of the cement was replaced with ground granulated blast furnace slag (GGBFS). The cement-slag-fine aggregate-coarse aggregate ratio was 1:1.1:4.87:6.55 (volumetric). The coarse aggregate used was limestone (Phelan et al., 1999). The chloride permeability of concrete from this site was investigated at five different maturity levels; 21, 28, 56, 113 and 365 days.

It was found that using pozzolanic materials (GGBFS), the permeability of concrete reduced to a large extent. From Fig. 3.10, it can be seen that current passed through 0-cure day concrete is much higher than any other concrete. This is because resistivity of chloride ion penetration grew more in cases of higher cured concrete. Also, almost horizontal current plots were observed in case of 2, 4, 8, 10 and 14 cure day concretes.

Fig 3.11 presents the El Paso 21-day permeability values in terms of charges passed through concrete cores with time. 0-cure day concrete shows high charges (equivalent to moderate permeability) whereas 14-cure day concrete shows low permeability. This again proves the beneficial effect of curing on permeability of concrete.

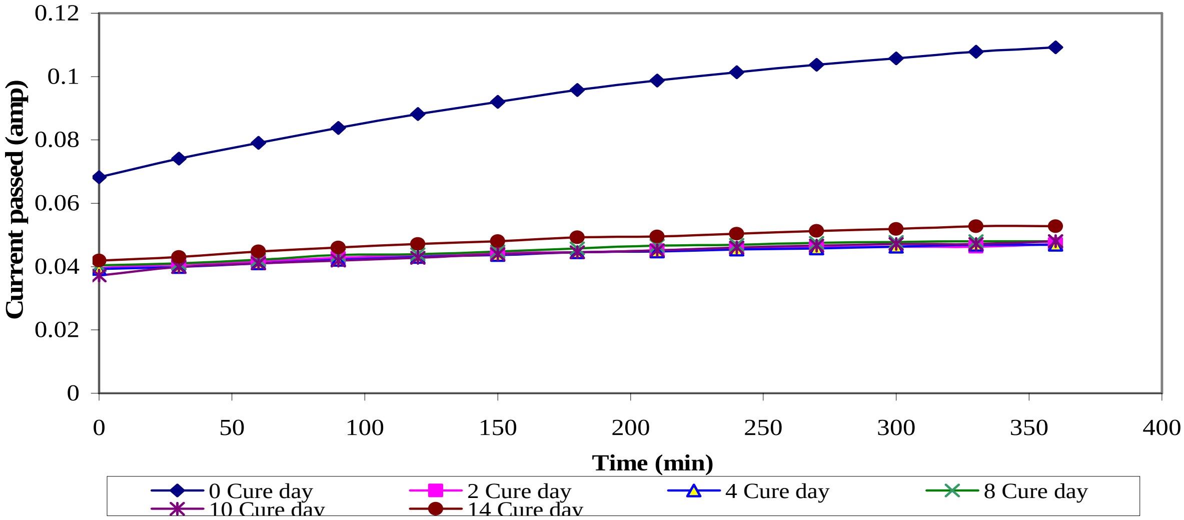


**Figure 3.10** Electrical Current Passed versus Time for El Paso 21-day Age Specimens

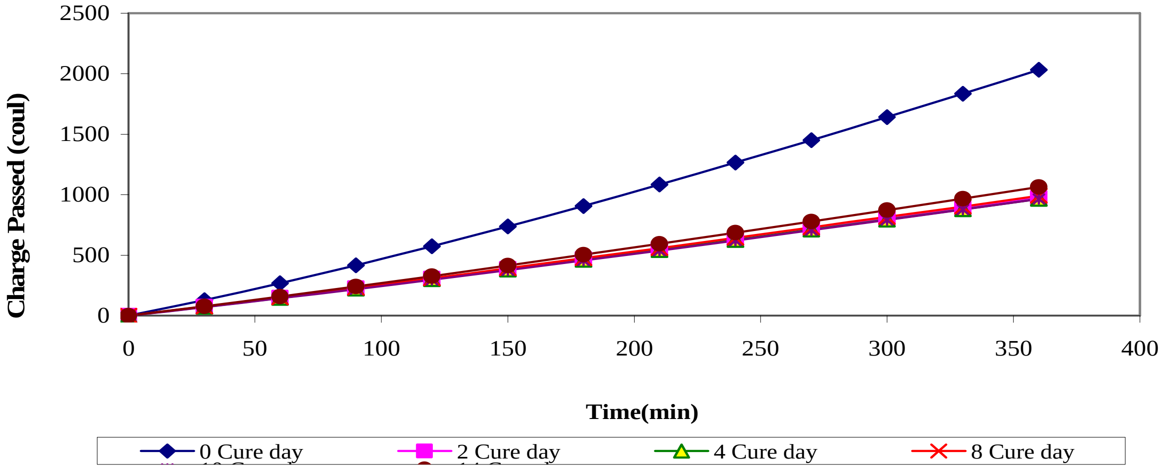


**Figure 3.11** Electrical Charge Passed versus Time for El Paso 21-Day Age Specimens

Fig. 3.12 shows El Paso 113-day age concrete permeability test results. It can be seen that for 2, 4, 8, 10 and 14-cure day concretes, current passed through concrete cores remain almost the same as the initial current reading, irrespective of time. Due to depletion of $OH^-$ ion concentration, the conductivity of concrete decreases, which is why the current reading did not change significantly.

Use of ground granulated blast furnace slag involved pozzolanic reaction in concrete. The higher the replacement percentage of ordinary Portland cement by GGBFS, the lower the pore fluid pH. In the case of El Paso mix design about 50% cement was replaced by GGBFS.

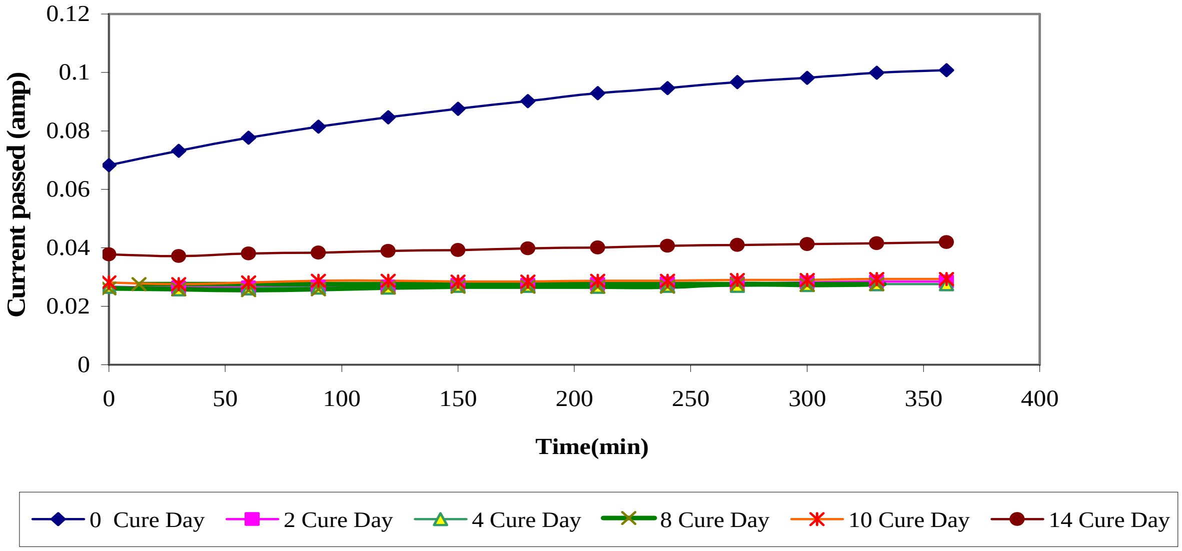


**Figure 3.12** Electrical Current Passed versus Time for El Paso 113-Day Age Specimens

During RCPT, the pore fluid pH influences the total charge passed through GGBFS concrete. Also, the influence of pore fluid pH on the charge passed is governed by the curing regimen as the $OH^-$ ions in the pore fluid deplete with the pozzolanic reaction. That’s the probable reason for greater reduction in the charges passed through GGBFS concrete.

The effect of age of concrete on permeability of concrete is evident in Fig. 3.13. Overall 365-day concrete showed very low permeability with low permeability in the case of 0-cure day concrete.

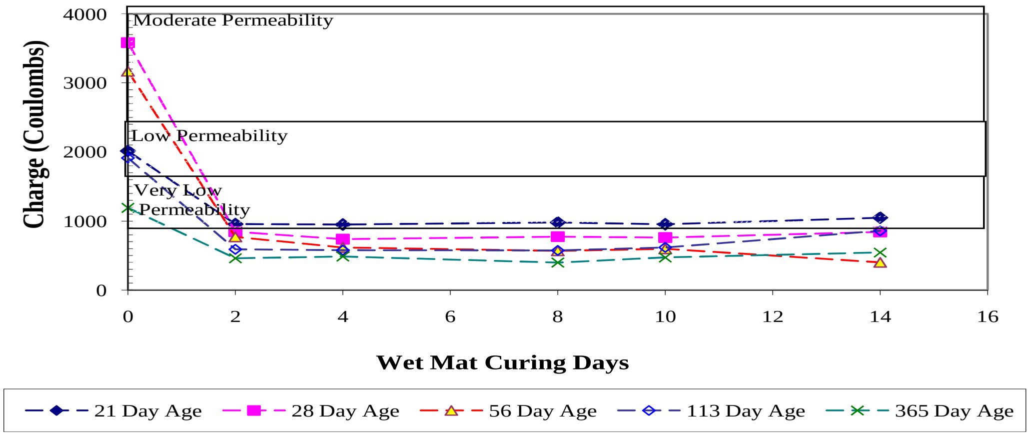


**Figure 3.13** Overall El Paso Permeability Test Results

Site temperature was not recorded before the permeability test of the El Paso 56-day age concrete. During the test run of El Paso 365-day concrete, temperature rose in the pore solution because heat evolved in concrete was too low.

**Table 3.3** Temperature of El Paso Permeability Cores (Maximum allowed 195°F)

| El Paso Test | Temperature after 360min test run (°F) | | | | | |
|---|---|---|---|---|---|---|
| | 0-cure day concrete | 2-cure day concrete | 4-cure day concrete | 8-cure day concrete | 10-cure day concrete | 14-cure day concrete |
| 56-day | 102 | 95 | 90 | 90 | 88 | 85 |
| 113-day | 93 | 85 | 84 | 82 | 80 | 85 |
| 365-day | 81 | 73 | 74 | 73 | 72 | 72 |

Fig. 3.14 clearly depicts the effect of curing on initial current passed through concrete. The 365-day-old concrete shows an approximately 37% decrease in initial current with respect to 56 and 113-day-old concrete, and shows a 44% decrease with respect to 21-day-old concrete after 14-day wet-mat curing.

Without any curing these values are: 32% decreases with respect to 21 and 113-day-old concrete. This reveals the fact that with increase in age of concrete, electrical conductivity of concrete has decreased.

Addition of pozzolanic material (GGBFS) increased the electrical resistivity of the concrete mixture. For a given proportion of GGBFS (50% replacement), the higher the period of wet mat curing the higher the electrical resistivity. Wet-mat curing for a longer duration (14 days) provided the maximum resistance to chloride penetration.

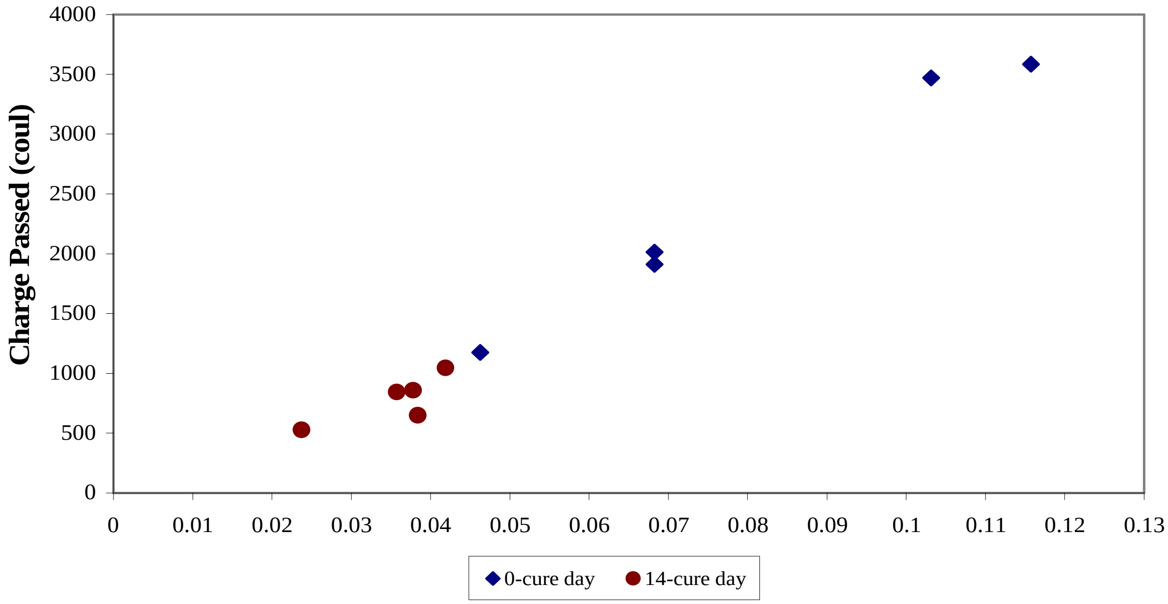


**Figure 3.14** Plot of Initial Current versus Charge Passed Through 0-Day and 14-Day Wet-Mat Cure Day Concrete for El Paso Site

*3.1.2.3 Fort Worth Test Slab*

For the Fort Worth site, concrete was mixed at a water-cement ratio of 0.43 and 25% cement was replaced with class F fly ash. The cement - fly-ash - fine aggregate - coarse aggregate ratio was 1:0.34:3.2:4.54 (volumetric). The coarse aggregate used was limestone (Phelan et al., 1999). Four different maturity levels of concrete were studied: 28, 56, 110 and 395-days.

In comparison with El Paso 28-day permeability test results, Fort Worth test results showed somewhat higher initial current in the cases of 2, 4, 8, 10 and 14-day cured concrete. However, little difference between initial and end current readings was observed in case of Fort Worth 28-day permeability test results. This may be due to the pozzolanic activity of fly ash.

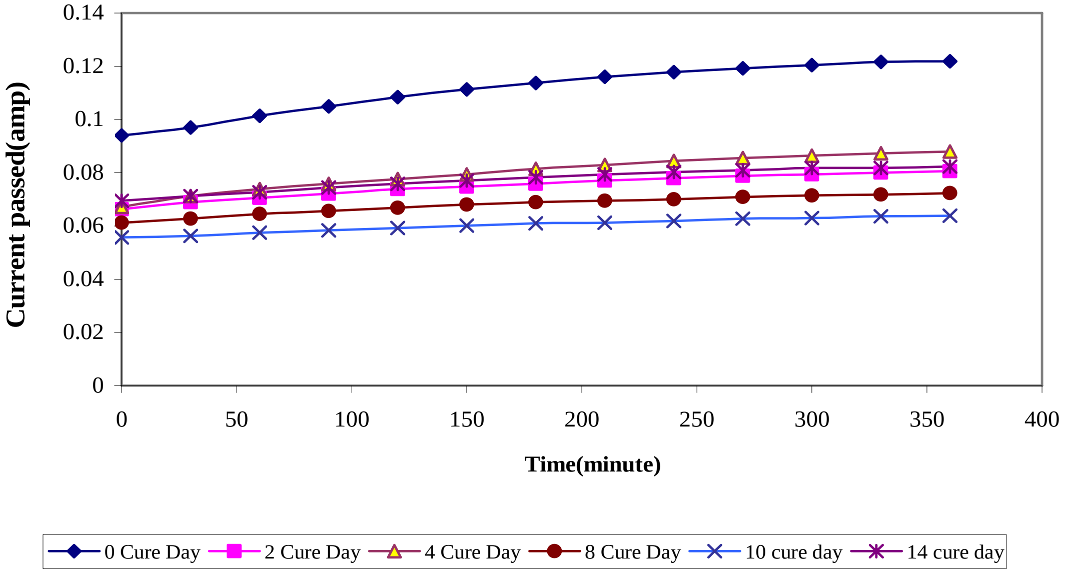


**Figure 3.15** Electrical Current Passed versus Time for Fort Worth 21-Day Age Specimens

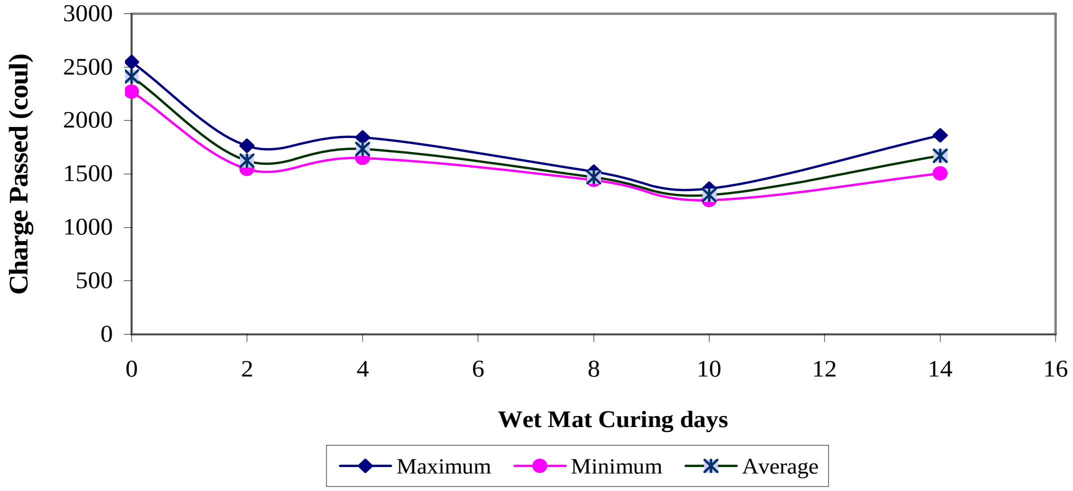


**Figure 3.16** Fort Worth 28-Day Permeability Test Result

Fig. 3.16 represents the average charges passed through Fort Worth 28-day concrete. Temperature rise in 0-cure day concrete was observed to be 26°F and that in 4-cure day concrete was 24°F.

No significant temperature rise was observed in other cure day concretes. 0-cure day concrete showed moderate permeability, whereas 14-cure day concrete showed low permeability, which reflects the effect of curing on permeability of concrete. Table 4.4 shows the temperature of the pore solution measured at the end of the RCPT.

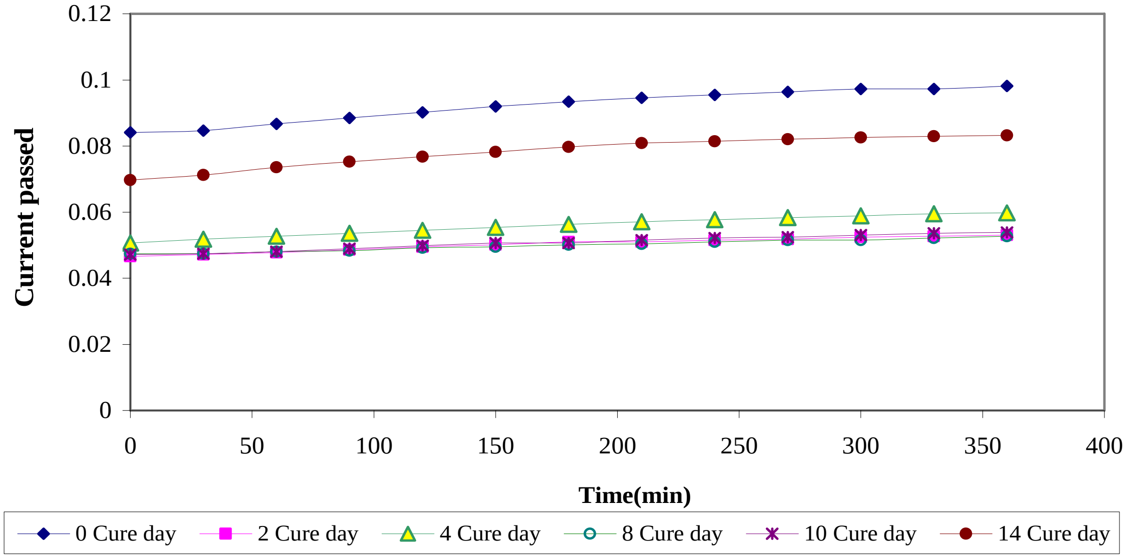


**Figure 3.17** Electrical Current Passed versus Time for Fort Worth 56-Day Concrete Specimens

Fort Worth 56-day permeability result showed lower initial current than 28-day permeability results. An interesting change in current readings was observed in the case of 14-cure day concrete. It was higher than current readings of 2, 4, 8 and 10-cure day concrete. This may be due to a rise in temperature during the 14-cure day concrete test run, which was about 31°F (Room temperature was 70°F). Consequently, the charge readings were also found to be higher than that of 2, 4, 8 and 10-cure day concretes (Fig.3.18).

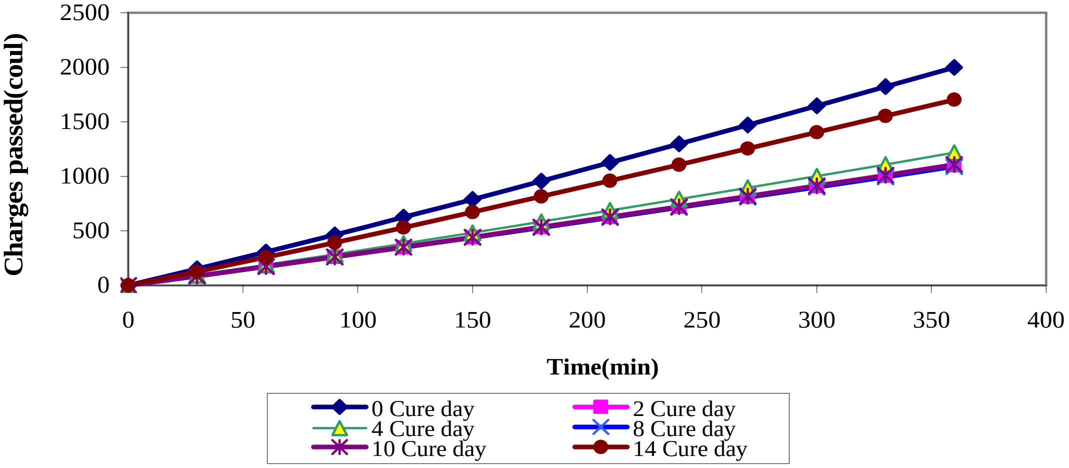


**Figure 3.18** Electrical Charge Passed versus Time for Fort Worth 56-Day Concrete Specimens

The overall Fort Worth 56-day permeability results were found to be low. 0-day and 14-cure day concrete showed low permeability, whereas the concrete of other cure regimens showed very low permeability. 14-cure day concrete showed permeability near the lower limit of low permeability range and 0-cure day concrete showed permeability near the higher limit of low permeability range.

**Table 3.4** Temperature of Fort Worth Permeability Cores (Maximum allowed 195°F).

| Fort Worth Test | Temperature after 360min test run (°F) | | | | | |
|---|---|---|---|---|---|---|
| | 0-cure day concrete | 2-cure day concrete | 4-cure day concrete | 8-cure day concrete | 10-cure day concrete | 14-cure day concrete |
| 28-day | 96 | 85 | 94 | 86 | 80 | 90 |
| 56-day | 107 | 85 | 92 | 90 | 90 | 101 |
| 110-day | 90 | 80 | 75 | 74 | 74 | 72 |
| 395-day | 79 | 74 | 72.5 | 73 | 73 | 73.5 |

Unlike 56-day permeability test results, Fort Worth 110-day permeability test results showed very reasonable effect of curing on permeability of concrete. The average ratio of 110-day permeabilities (in terms of charges passed) for the 14, 10, 8, 4, and 2 to 0-day wet mat cured concrete was respectively 0.44, 0.43, 0.454, 0.475, and 0.49. Only 2 days of curing reduced about 49 percent permeability of concrete. Without curing and under the action of air-drying the 110-day concrete showed moderate permeability, whereas 14-day curing provided very low permeability to fly ash concrete.

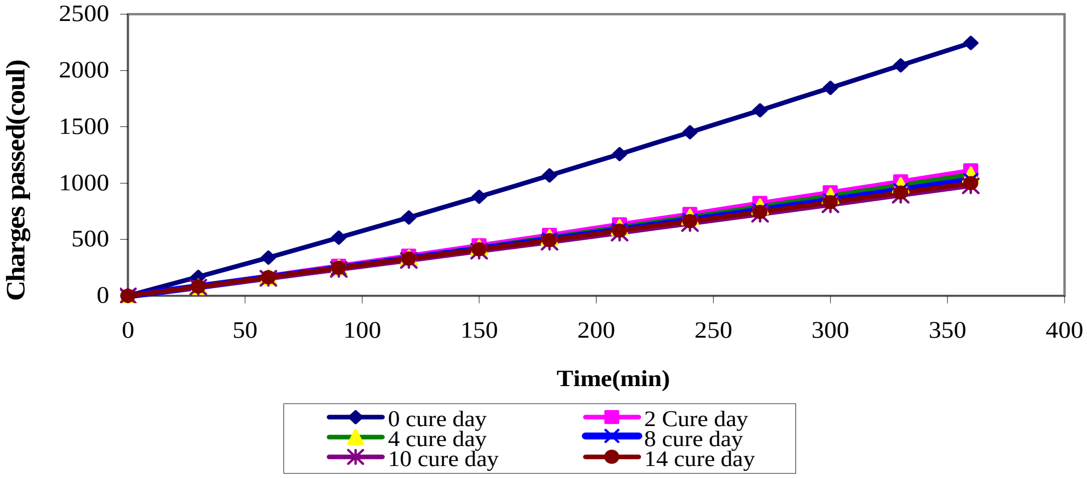


**Figure 3.19** Electrical Charge Passed versus Time for Fort Worth 110-Day Concrete Specimens

Fig. 3.20 represents the effect of curing and age of concrete on the initial current and charges found from RCPT result of Fort Worth concrete specimen. While 28-day concrete with no curing showed high initial current as well as charges, 28-day concrete with 4-days of curing and 14-days of curing showed low initial current and charges. With 4 and 14 days of curing, there were approximately the same initial current and charges. It was observed that both the 4- and 14-day cured concrete showed similar low initial current and charge at age of 28 days. At 110 days, these results reduced to a lower value. This reflects the fact that with increase in age of concrete, there is a probability of greater reduction in permeability of concrete.

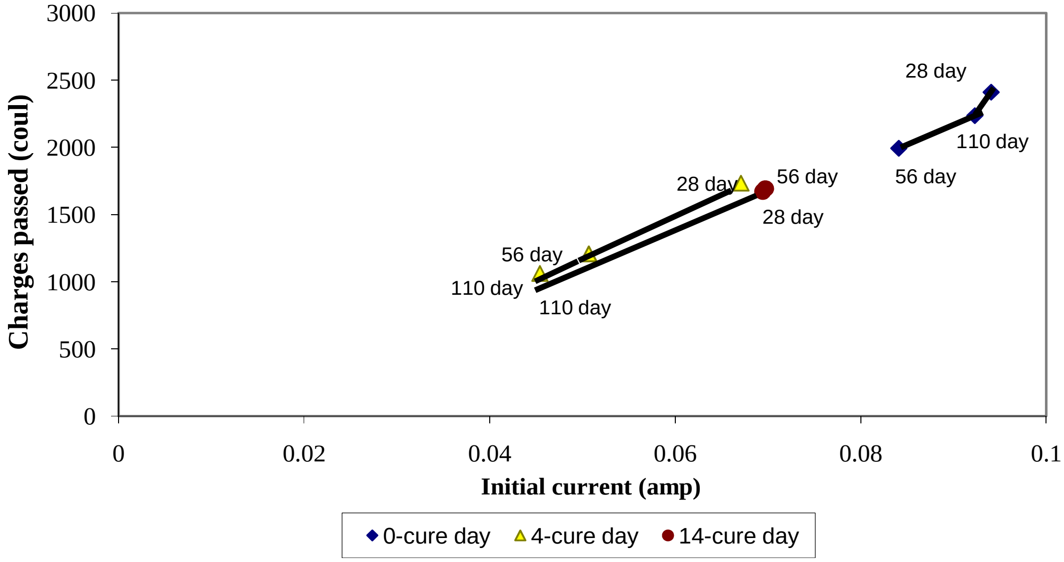


**Figure 3.20** Plot of Initial Current versus Charge Passed Through 0, 4 and 14-Wet Mat Cure Day Concrete for Fort Worth Site

Fig. 3.21 depicts the effect of extended curing on different aged concrete permeability. Though the 28 and 56-days aged concrete resulted in similar charges even after 14-days of curing, it reduced to a larger extent at 110 days.

From the experimental test results, it has been found that class F fly ash is very effective in reducing the permeability of concrete even at a lower rate of replacement level (25%). This could be explained by the pozzolanic reaction caused by fly ash. Fly ash is capable of reacting with the calcium hydroxide produced during cement hydration and forms calcium silicate hydrate. This silicate hydrate fills large capillary voids and their continuity is disrupted. Consequently, the blocking of capillary voids reduces the permeability of concrete.

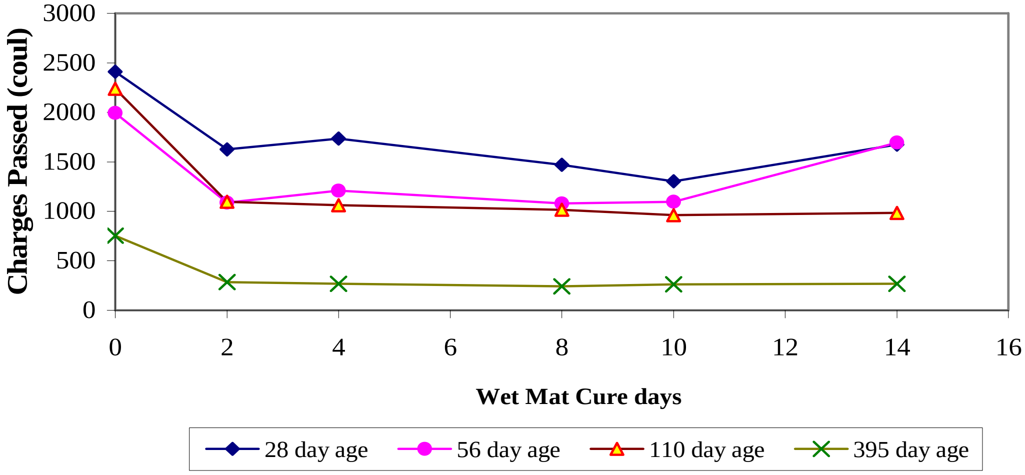


**Figure 3.21** Overall Fort Worth Permeability Test Result

*3.1.2.4 San Antonio Test Slab*

Permeability tests were conducted on 56-day and 153-day aged concrete specimens from the San Antonio site. For this site, concrete was mixed at a water-cement ratio of 0.44 and 21 percent of cement was replaced with class C fly ash. The cement- fly ash -fine aggregate-coarse aggregate ratio was 1:0.31:2.71:4.49 (volumetric). The coarse aggregate used was limestone (Phelan et al., 1999).

From the plot of current reading versus time (Fig. 3.22), it was found that initial current results were lower than any other concrete from other sites.

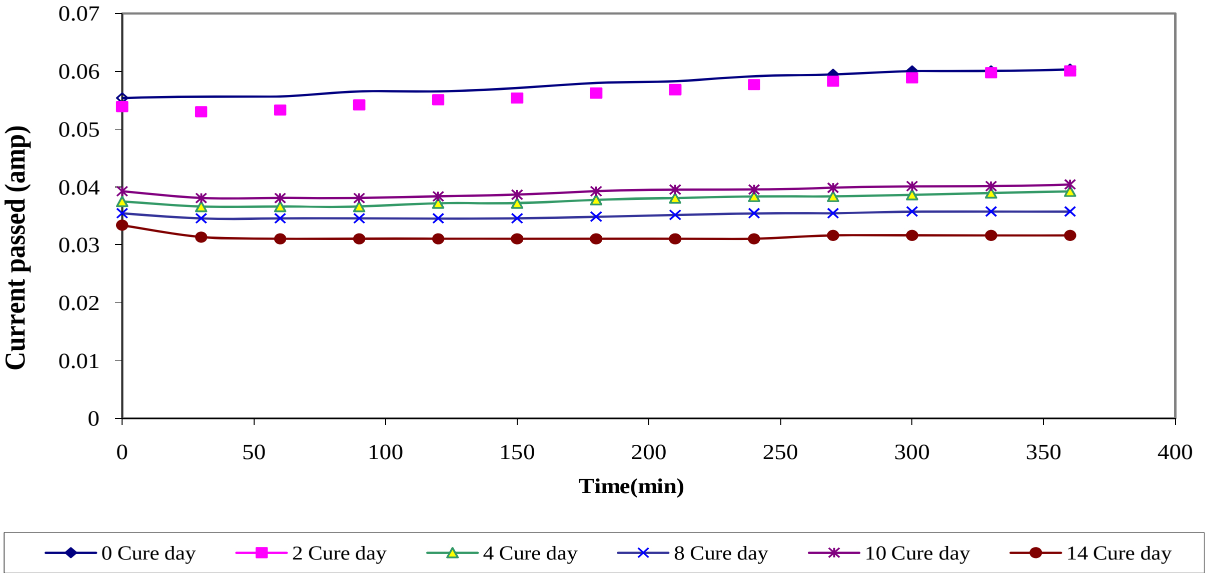


**Figure 3.22** Electrical Current Passed versus Time for San Antonio 56-Day Age Specimens

An exceptional interesting matter was observed in case of San Antonio 56-day concrete. Initial current slightly decreased after some time had elapsed and it did not increase a significant amount. An almost horizontal curve signifies the fact that resistivity of concrete was too high due to depletion of hydroxyl ion conductivity.

Among the different curing regimes, 0- and 2-cure day concrete showed higher current readings as well as charges. San Antonio 56-day permeability results showed low permeability of concrete; even without curing it was in the range of low permeability.

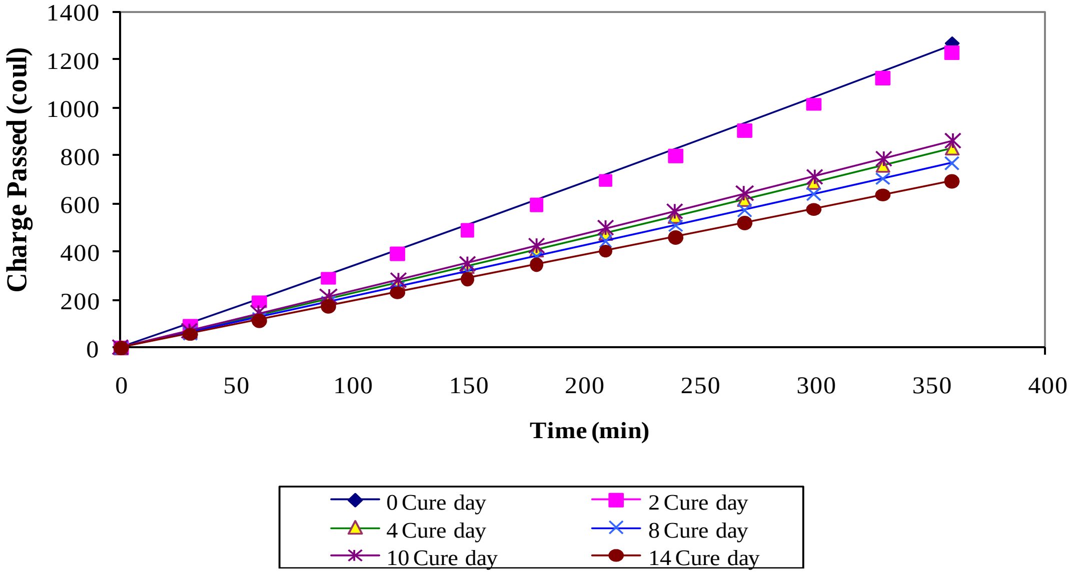


**Figure 3.23** Electrical Charge Passed versus Time for San Antonio 56-Day Age Specimens

After 14 days of moist curing this value reduced to a minimum result, which is in the range of very low permeability.

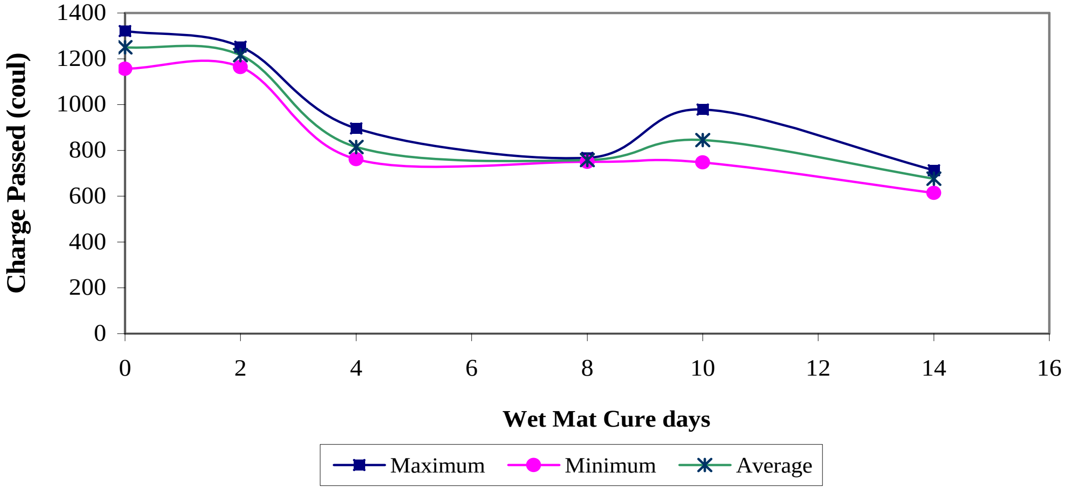


**Figure 3.24** San Antonio 56-Day Permeability Test Result

Temperature rise during the test was very little. It was observed that after a 360-minute run of the test, both 0- and 2-cure day concrete showed similar temperature rises, which reflects the fact of similar current reading.

**Table 3.5** Temperature of El Paso Permeability Cores (Maximum allowed 195°F).

| San Antonio Test | Temperature after 360min test run (°F) | | | | | |
|---|---|---|---|---|---|---|
| | 0-cure day concrete | 2-cure day concrete | 4-cure day concrete | 8-cure day concrete | 10-cure day concrete | 14-cure day concrete |
| 56-day | 81 | 81.5 | 76.5 | 75.5 | 76.5 | 74.5 |
| 153-day | 83.5 | 78 | 75 | 76 | 77 | 78 |

From Fig. 3.24, it can be seen that only 4 days of curing significantly reduced the current passing values.

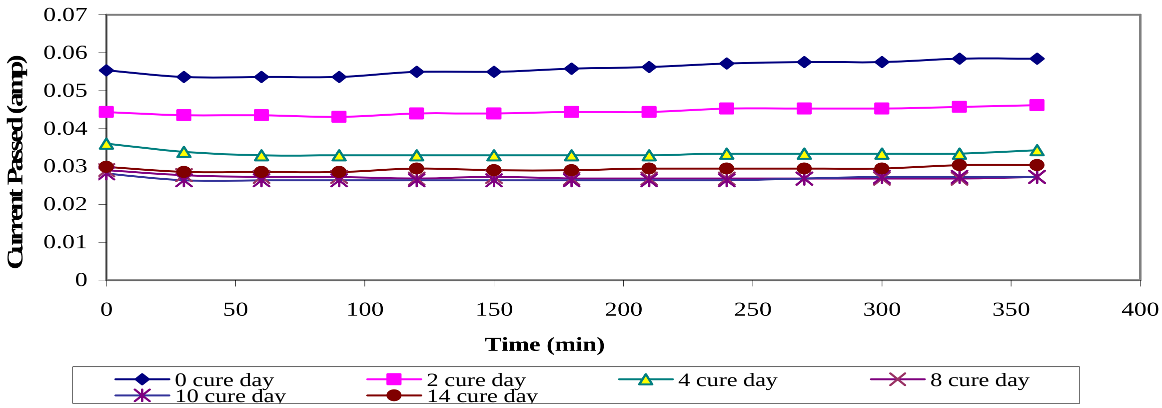


**Figure 3.25** Electrical Current Passed versus Time for San Antonio 153-Day Age Specimens

The average ratio of 56-day permeabilities (in terms of charges passed) for the 14,10, 8, 4, and 2 to 0-day wet mat cured concrete were respectively 0.54, 0.68, 0.61, 0.65, and 0.97.

#### *3.1.2.5 Houston Test Slab*

After the first five sites were cast, a decision was made to only conduct the RCPT test on cores taken at 56 days. Houston district cores were taken at 56 days only. For this site, concrete was mixed at a water-cement ratio of 0.46 and 27 percent of cement replacement with class C fly ash. The cement- fly ash -fine aggregate-coarse aggregate ratios were 1:0.31:2.71:4.49 (volumetric). The coarse aggregate used was limestone.

Figure 3.26 on the following page shows the current flow through the concrete specimen during the 6-hour RCPT test. The results do not follow the expected pattern, but can be explained. The ready-mixed concrete used for this test slab came from two different concrete trucks. No noticeable difference was observed between the mixes at the time of casting and both truckloads were ordered for the same standard mix design. However, subsequent tests indicated that concrete from the second truck that went to cast the 0-day cured panel and a part of the 2-day cured panel was stronger than the concrete from the first truck. It appears that this anomaly in the concrete mixes is reflected in the RCPT current flow results. The 0-day and 2-day cure core specimens indicated the lowest current flow (thus lower permeability) when compared to other curing durations. Furthermore, the cores from the 14-day cure panel showed the highest current flow. For this panel, only two replicate specimens were tested in the RCPT because the third specimen was damaged during handling. Furthermore, the two specimens showed significant variability in their current flow values. As expected, the electric charges for the specimens indicated a similar trend since the charge was calculated using the same electric current values. The chart for electric charge is displayed in Figure 3.27 on the following page.

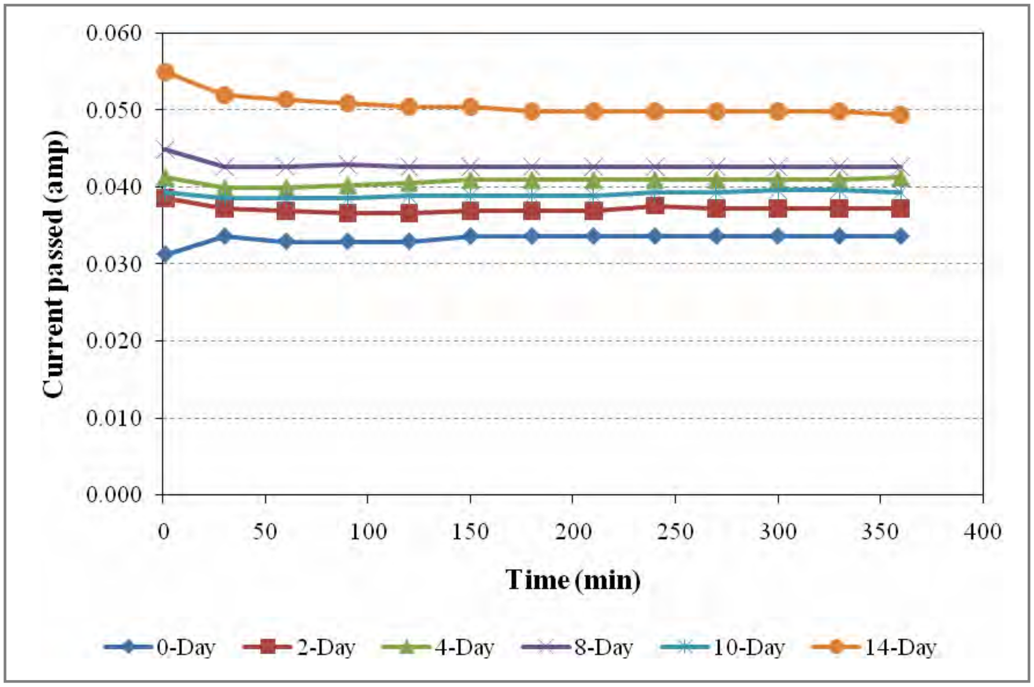


**Figure 3.26** Electrical Current Passed versus Time for Houston 56-Day Age Specimens

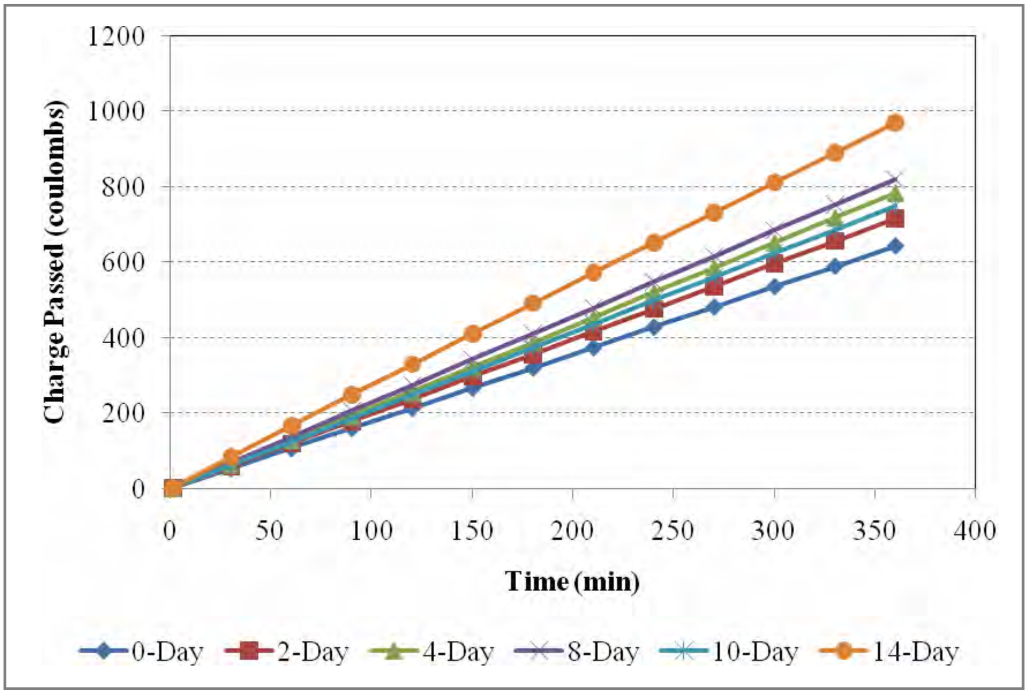


**Figure 3.27** Electrical Charge Passed versus Time for Houston 56-Day Age Specimens

A summary of the 56-day specimen test data is presented in Figure 3.28 below. The anomalies in the results associated with the two concrete mixes used are clearly visible in this chart. The chart is presented in such a way that the variability of the three replicate test results can be compared along with their average values.

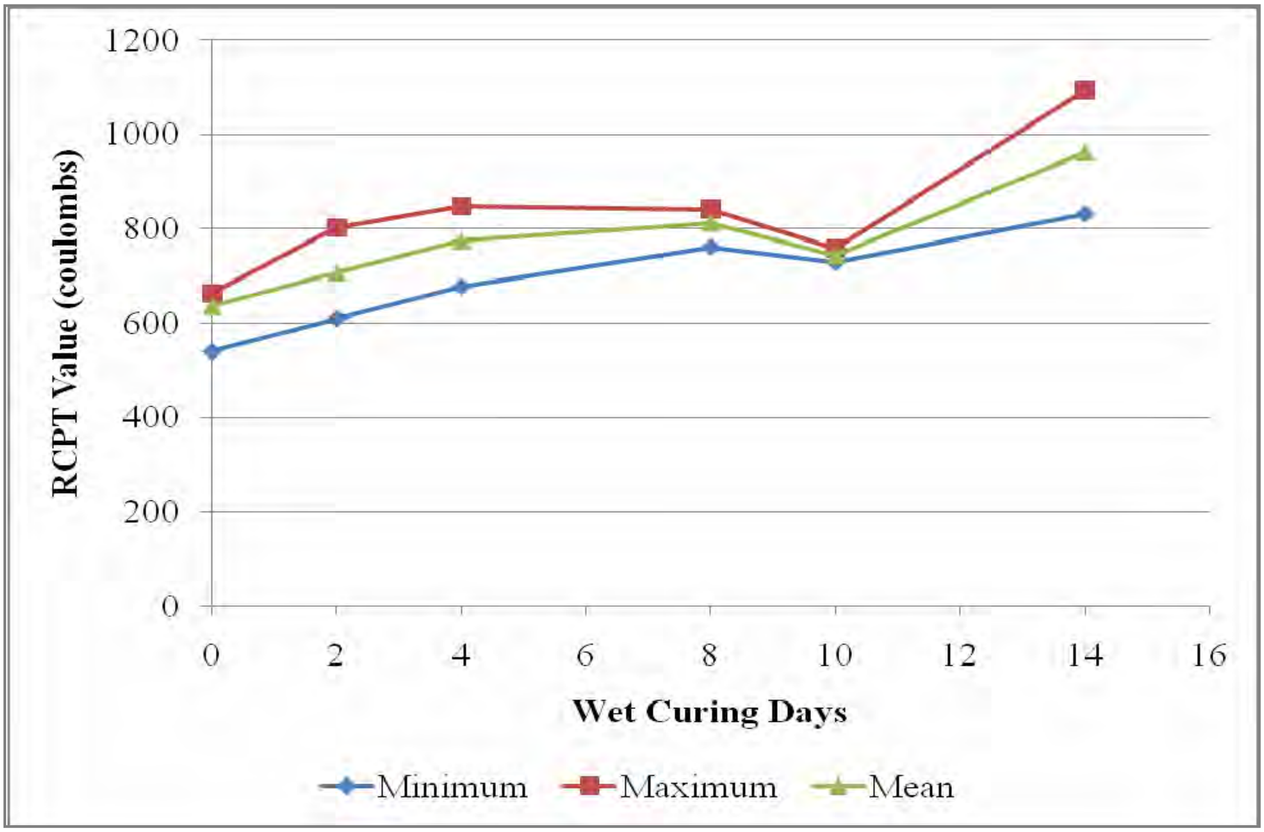


**Figure 3.28** Houston 56-day RCPT Test Results

*3.1.2.6 Atlanta District Mix Design*

No test slab was constructed in the Atlanta and Pharr districts. However, a mix design was selected from each of these two districts and their constituent materials were transported to Lubbock from the same suppliers who typically supply these materials. Since no field test slabs were constructed for these two mix designs, instead of field cores, test specimens for the RCPT were taken from laboratory cast 4-inch diameter, 8-inch long cylindrical specimens that were cast using local materials from those districts. The 2-inch specimen disks required for the test were saw-cut at 56-day age from the longer cylinders. For the Atlanta mix design, concrete was mixed at a water-cement ratio of 0.46 and 21 percent of cement replacement with class F fly ash. The cement- fly ash -fine aggregate-coarse aggregate ratios were 1:0.31:2.71:4.49 (volumetric). The coarse aggregate used was siliceous river gravel.

Figure 3.29 shows the results of electric current flow through the concrete specimen during the 6-hour RCPT test. The 0-day and 2-day cured specimens show the higher current flow, indicating the highest levels of chloride permeability. This was expected and is consistent with the observations made from the first four mix designs. An interesting departure from the first four sites is the trend in current flow with time during the six-hour RCPT test. The current flow results from the four sites shown previously indicated a gradually increasing current flow with time during the six-hour test. However, results from the Atlanta mix design for the 0-day cure specimens showed current flow decreasing after approximately 4 hours of the test and for the 2-day cure specimens, current flow decreased after 4.5 hours. The Atlanta mix design was one of two mix designs that used siliceous river gravel coarse aggregate. These two mix designs showed the highest RCPT values (moderate RCPT rating) when compared to the other mixes tested. This could be due to the siliceous gravel course aggregate used in these two mixes, which is known to result in weak bonding between the aggregate and the cement paste, thus increasing the permeability of concrete due to preferred flow paths along the interfaces. Therefore, it is possible to postulate that crystallized calcium hydroxide present on the aggregate surfaces may result in increased pozzolanic reaction on them between calcium hydroxide crystals and class F fly ash in the mix. Such reactions are likely to densify the aggregate-cement interface with newly formed calcium silicate hydrate resulting in lower permeability as the test progresses. A problem was encountered with the 10-day cure specimens during the curing process, and those specimen were not included in Figures 3.29 and 3.30 (electric charge).

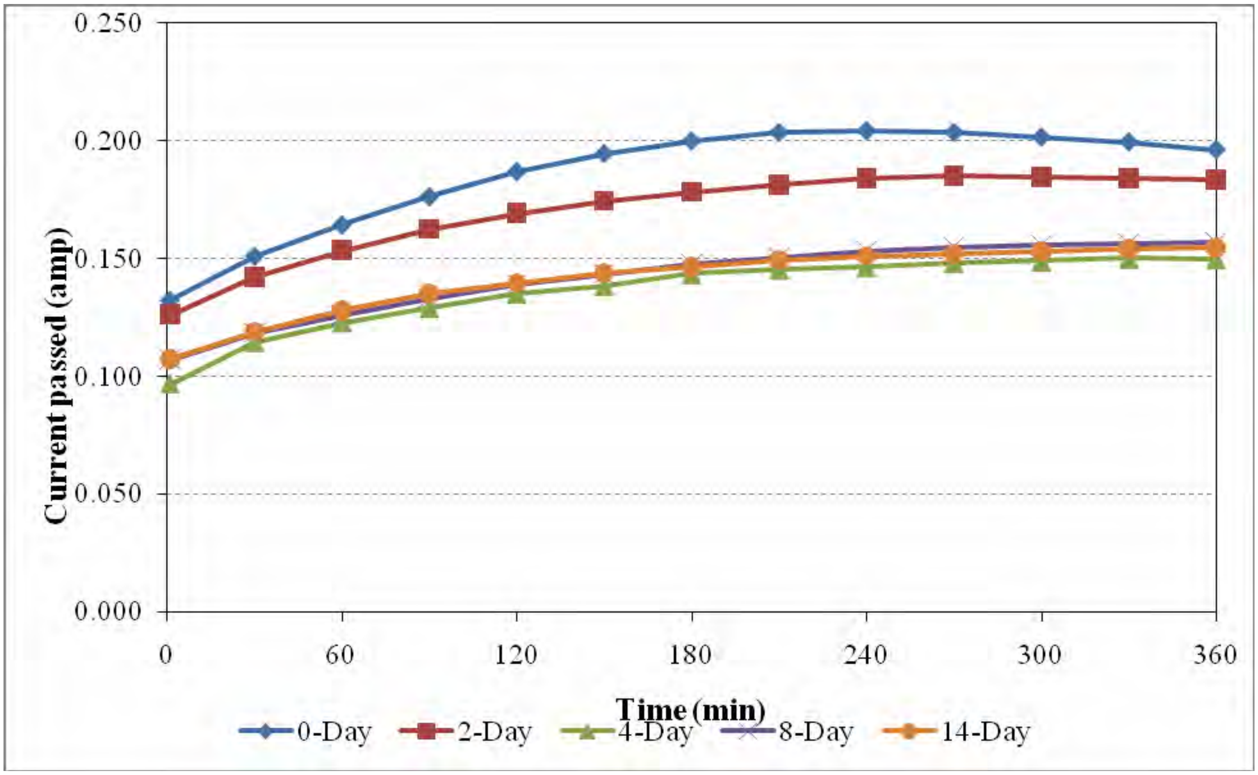


**Figure 3.29** Electrical Current Passed versus Time for Atlanta 56-Day Age Specimens

A summary of the 56-day specimen test data is presented in Figure 3.31 on the following page. The three curves correspond to the statistical data for the three replicate specimens tested for a particular curing duration. Improvements in the resistance to chloride permeability due to extended curing were observed up to 4 days of curing, beyond which the RCPT values remained practically constant. As mentioned previously, data for the 10-day curing specimens were not included.

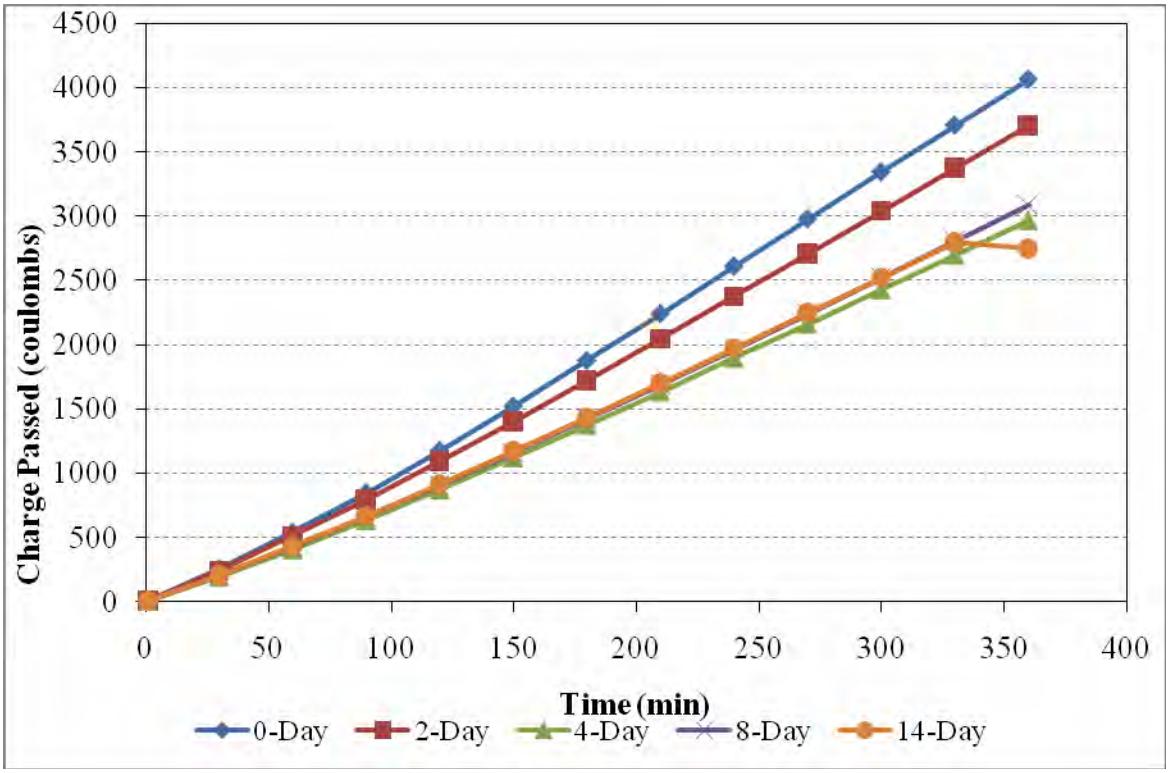


**Figure 3.30** Electrical Charge Passed versus Time for Atlanta 56-Day Age Specimens

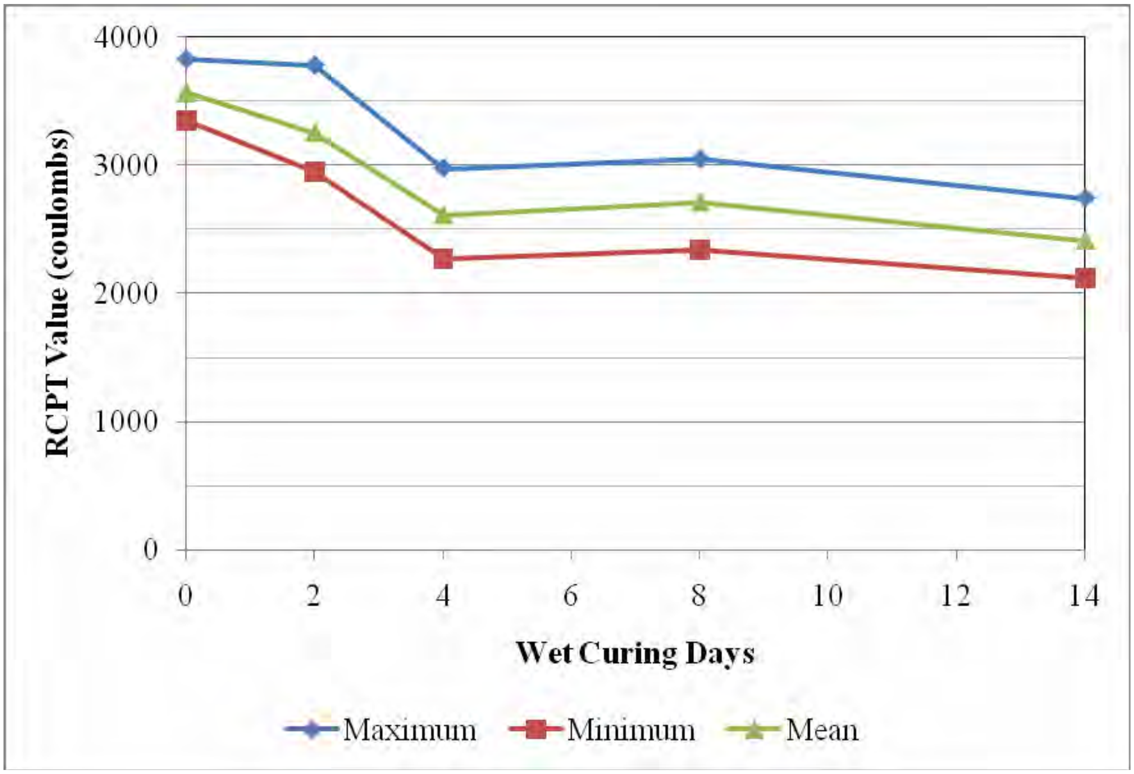


**Figure 3.31** Atlanta 56-Day RCPT Test Results

### *3.1.2.7 Pharr District Mix Design*

As indicated above, no test slab was constructed in the Pharr district. However, a mix design was selected from that district and its constituent materials were transported to Lubbock from the same suppliers who typically supply these materials. Test specimens for the RCPT were taken from laboratory cast 4-inch diameter, 8-inch long cylindrical specimens that were cast using local materials from those districts. For the Pharr mix design, concrete was mixed at a water-cement ratio of 0.41 with no cement replacement with fly ash. The cement- fly ash -fine aggregate-coarse aggregate ratios were 1:0.31:2.71:4.49 (volumetric). The coarse aggregate used was siliceous river gravel.

Figure 3.32 shows the results of electric current flow through the concrete specimen during the 6-hour RCPT test. The 0-day, 2-day and 4-day cured specimens show higher current flows, indicating the highest levels of chloride permeability. This was expected and is consistent with the observations made from the first four mix designs. A significant difference was observed between the 4-day and 8-day specimen results. The 14-day specimen results are not shown in this chart due to problems encountered during the data collection process. The corresponding charge data are shown in Figure 3.33 showing trends similar to that of the current flow.

A summary of the 56-day specimen test data is presented in Figure 3.34 on page 43. The three curves correspond to the statistical data for the three replicate specimens tested for a particular curing duration. Improvements in the resistance to chloride permeability due to extended curing were observed up to 10 days of curing, and this observation was different compared to the other mixes tested. One notable aspect of this mix design was no pozzolanic material was used in this mix. The Pharr District mix used Type I ordinary Portland cement along with siliceous river gravel coarse aggregate.

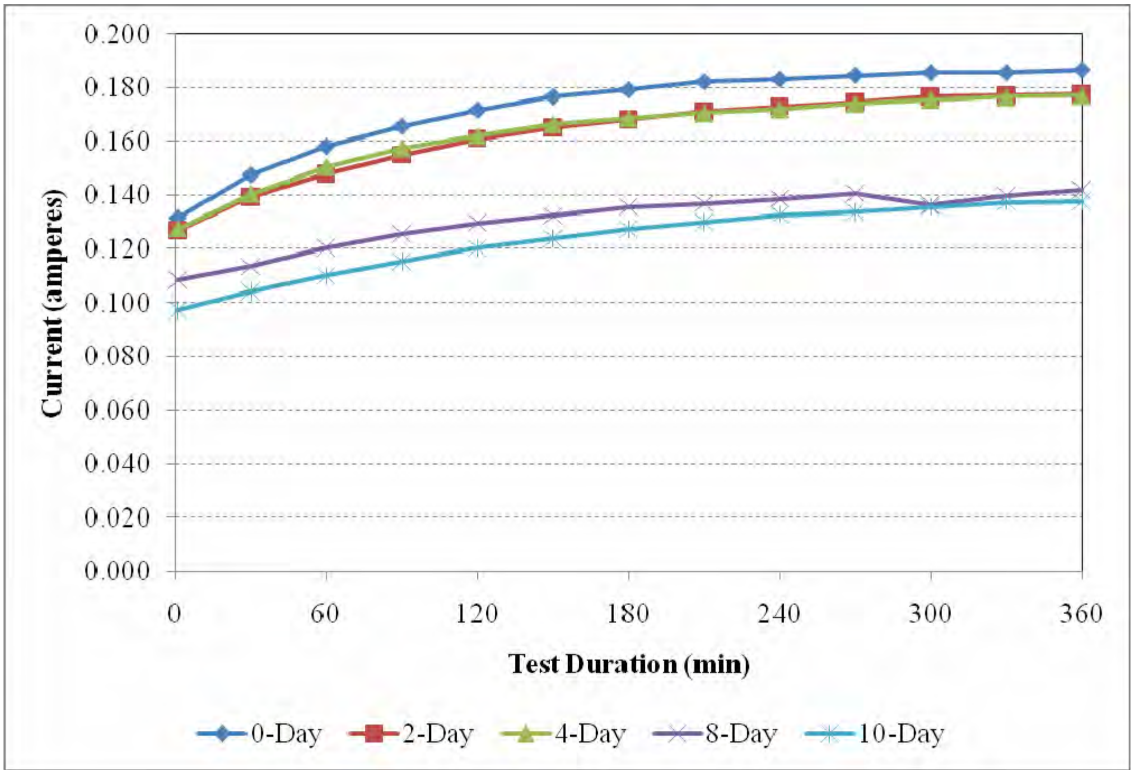


**Figure 3.32** Electrical Current Passed versus Time for Pharr Mix 56-Day Age Specimens

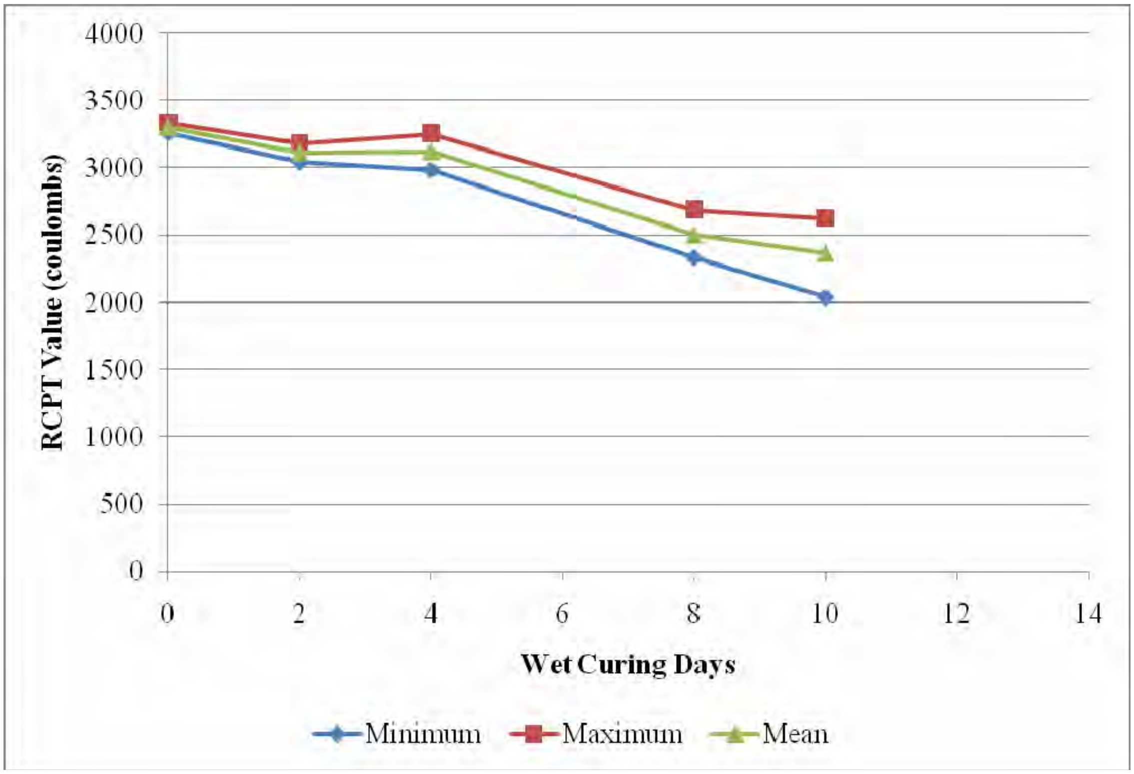


**Figure 3.33** Electrical Charge Passed versus Time for Pharr Mix 56-Day Age Specimens

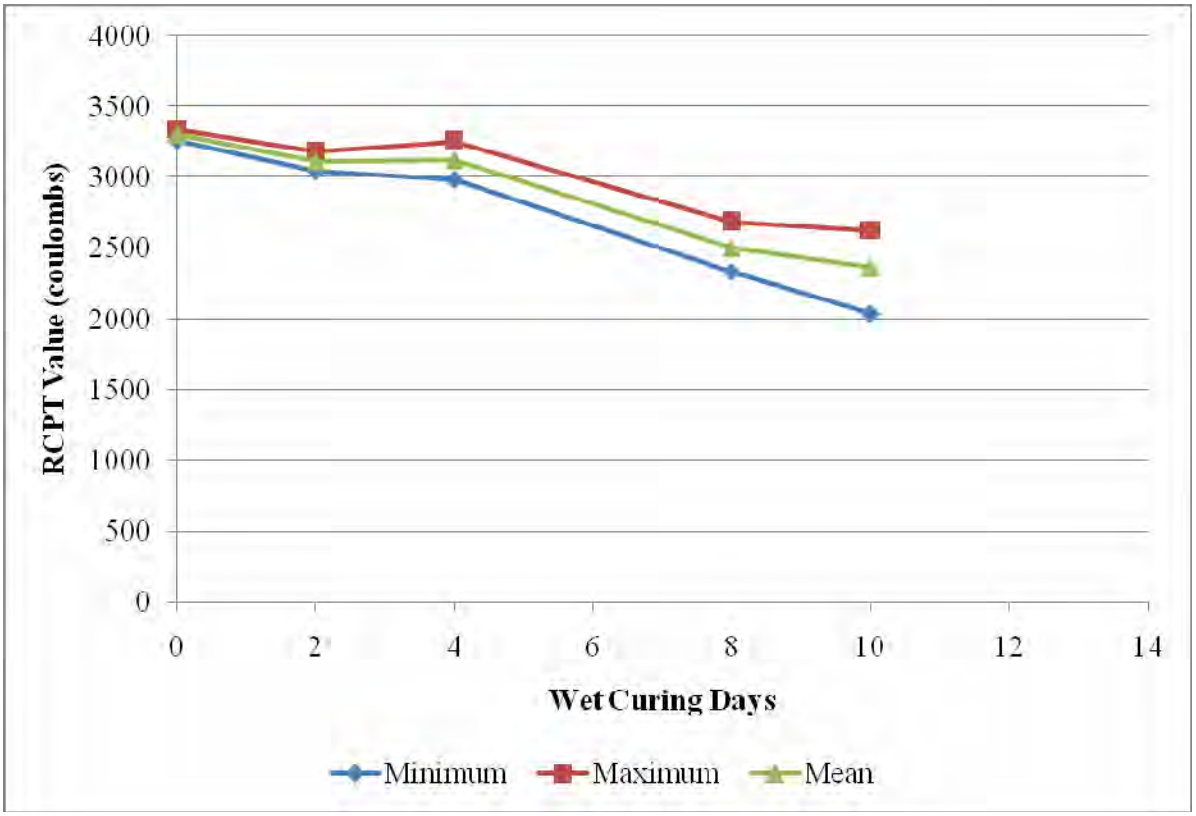


**Figure 3.34** Pharr Mix 56-day RCPT Test Results

### 3.1.3 Parametric Study for Prediction of Chloride Penetration

Determination of diffusion coefficient of concrete is possible from the laboratory test results by modifying the results obtained under standard conditions. Standard conditions are typically considered to be a temperature of 23°C, a relative humidity of 100%, and 28 days of hydration under standard temperature and relative humidity. This reference value of the diffusion coefficient will not be applicable for different ambient climatic condition. According to Saetta et al. (1993) this reference value can be corrected assuming that temperature, maturation time and relative humidity are the only variables that affect the diffusion coefficient in different climatic conditions.

The correction is as follows:

$$D_i = D_{i,ref} \cdot f_1(T) . f_2(t_e) . f_3(h) \qquad (3.1)$$

Where, $f_1(T)$ account for the dependence of temperature, $f_2(t_e)$ is the effect of maturation age of concrete and $f_3(h)$ is the effect of pore humidity on intrinsic diffusion coefficient, $D_i$..

$f_1(T)$ account for the variation of intrinsic diffusion coefficient with the temperature $T$. According to Arrhenius' law;

$$f_1(T) = \exp\left[\frac{U}{R}\left(\frac{1}{T_0} - \frac{1}{T}\right)\right] \quad (3.2)$$

Where,

$T$ and $T_0$ are actual and reference temperature expressed in degree Kelvin,

$R$ is the gas constant in KJ / (mol. K), and

$U$ is the activation energy of the diffusion process (KJ/mol).

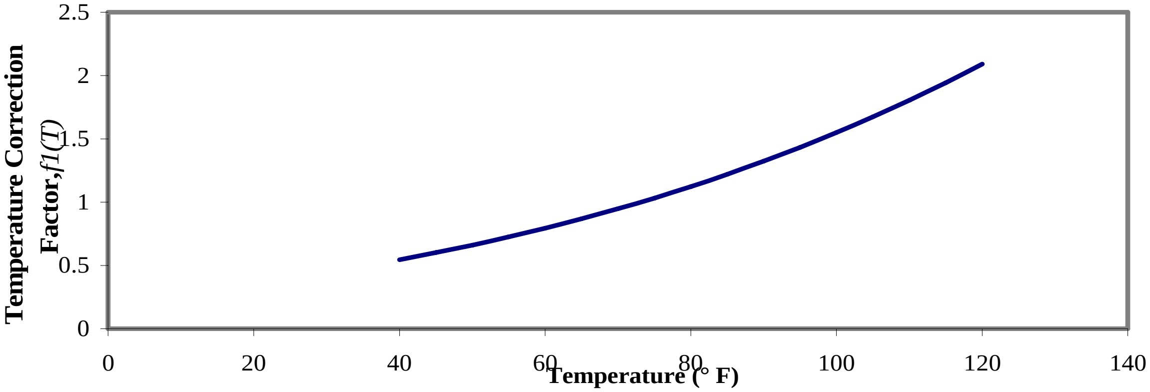


**Figure 3.35** Temperature Correction Factor for Different Ambient Temperatures

Fig. 3.35 shows that at the reference temperature ($T_o$) of 73°F, the temperature correction factor $f_1$ *(T)* is equal to 1.0. Also, the correction factor increases with increasing temperature from the reference temperature of 73°F (296°K).

Chloride diffusivity in concrete is also a function of the degree of hydration. Due to the decrease in interconnectivity with increasing hydration, chloride diffusivity decreases increasing cement hydration and reaches an asymptotic value when hydration nears completion. Bazant (1972) proposed that degree of hydration could be expressed as a function of the 'equivalent maturation time' $t_e$ defined by eq. 3.3.

$$t_e = t_{e,in} + \int_0^t \beta_h . \beta_t . dt \quad (3.3)$$

Where,

$$\beta_h = \left[1 + (3.5 - 3.5.h)^4\right]^{-1} \tag{3.4}$$

and, $$\beta_T = \exp\left[\frac{U}{R}.\left(\frac{1}{T_0} - \frac{1}{T}\right)\right] \tag{3.5}$$

in which $U/R$ = 2700°K and $T_0$ =296°K.

Saetta et al. (1993) also developed a relationship to model the decrease in diffusivity with increasing degree of hydration (Eqn 3.6).

$$f_2(t_e) = \varsigma + (1 - \varsigma).\left(\frac{28}{t_e}\right)^{1/2} \; ; \tag{3.6}$$

Where, $t_e$ are in days.

Coefficient $\zeta$ is the ratio between the diffusion coefficients for $t_e \rightarrow \infty$ and $t_e$ at 28 days. The range of its variation is from 0 to 1. This parameter ($\zeta$) measures how much diffusivity decreases with time. For concrete with low water-cement ratios and rapid hardening cement, which reaches low diffusion coefficient after a few days of hydration, the parameter $\zeta$ can be assumed to be approximately 1. For concrete with high water-cement ratios, this ratio will be lower than 1 because intrinsic diffusion coefficient decreases with equivalent maturation time at a more rapid rate (Saetta, 1993).

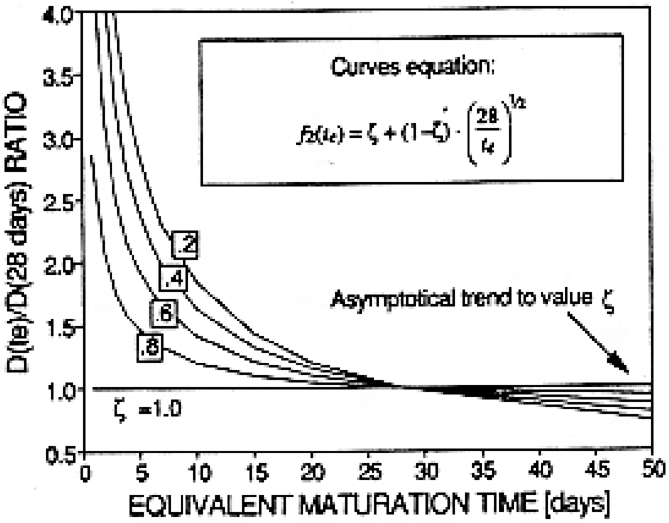


**Figure 3.36** Plot of Decrease of Chloride Diffusion Coefficient on Equivalent Maturation Time $t_e$ for Different Values of Parameter $\zeta$ (Saetta, 1993)

From the equation presented by Saetta et al. (1993), the correction factor for different concrete ages can be determined. Some of those values are shown in Table 3.6 below. Fig. 3.36 and Table 3.6 show that with increasing age of concrete the diffusivity of concrete decreases if two other variables (T and RH) are assumed to be constant. At early age the diffusivity will be high, which can be experimentally proven from the RCPT results. With minimum water/cement ratio, this factor will be uniform. For a given maturation time, the correction factor will be higher for high water/cement than that with lower water/cement ratio.

**Table 3.6** Correction of Chloride Ion Diffusivity of Concrete at Different Concrete Ages

| Age of concrete (days) | Ratio between the diffusion coefficients with respect to 28 days of age, ζ (derived from Eq. 6.6 and Fig. 6.2) | | | | |
|---|---|---|---|---|---|
| | 0.2 | 0.4 | 0.6 | 0.8 | 1 |
| 14 | 1.414 | 1.25 | 1.166 | 1.083 | 1 |
| 21 | 1.155 | 1.093 | 1.062 | 1.031 | 1 |
| 28 | 1 | 1 | 1 | 1 | 1 |
| 56 | 0.77 | 0.83 | 0.883 | 0.941 | 1 |
| 90 | 0.65 | 0.74 | 0.823 | 0.91 | 1 |
| 110 | 0.604 | 0.703 | 0.802 | 0.901 | 1 |
| 113 | 0.598 | 0.699 | 0.799 | 0.899 | 1 |
| 365 | 0.42 | 0.57 | 0.711 | 0.855 | 1 |

The effect of relative humidity on diffusion coefficient can also be modeled using the relationship in Eq. 3.7 (Bamforth et al., 1987).

$$f_3(h) = \left[1 + \frac{(1-h)^4}{(1-h_c)^4}\right]^{-1} \tag{3.7}$$

Where, $h_c$ characterizes the humidity at which the intrinsic diffusion coefficient, $D_i$ drops halfway between its maximum and minimum values. From the S-shaped curve (Fig. 3.37), it was found that when the relative humidity $h$ drops below the critical value $h_c$ = 75% (for 25°C), the ion diffusivity shows a rough drop.

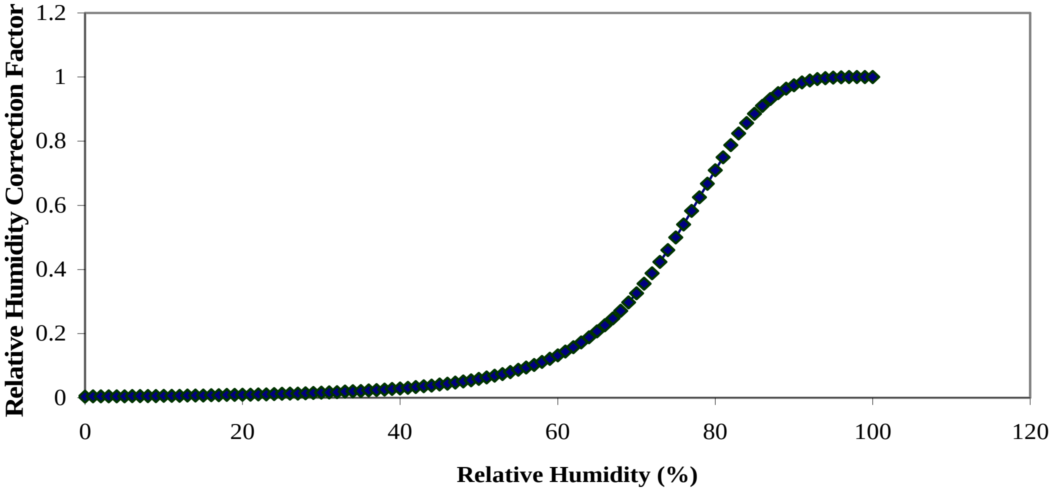


**Figure 3.37** Correction Factor for Different Relative Humidity (plotted using Eq. 3.7).

Laboratory tests were performed on fully-saturated concrete specimen. However, relative humidity is much lower under field conditions and with the decrease in relative humidity; the correction factor will be less than that for 100% relative humidity.

If the correction factors for temperature and relative humidity, as shown by equations 3.2 and 3.7, were combined it would take the shape shown in Fig. 3.38. For a given age of concrete, the correction for temperature and relative humidity will be such that at lower relative humidity values, (for example, 20%) the correction factor would lead to negligible diffusivity irrespective of the prevailing temperature.

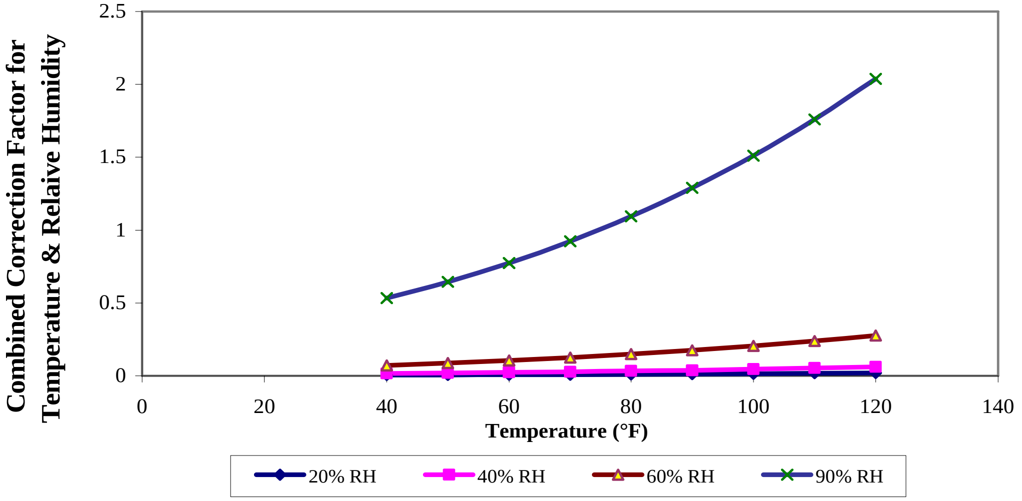


**Figure 3.38** Plot of Combined Effect of Temperature and Relative Humidity (%) on the Diffusivity of Concrete

On the other hand, when the relative humidity is high (90 percent), at higher temperature (more than 73°F), the diffusivity of concrete will be higher at a specific concrete maturation time. If the temperature rises above 100°F at 90% RH, the diffusivity will be approximately twice the value at standard condition, whereas at low temperate regions or in cool weather, (for example, 40°F) this diffusivity value would be approximately half the reference value (at standard condition).

The effect of these parameters on different concretes located at various sites will be different depending on the age of concrete and ambient climatic condition. The combined effect of temperature, maturation time and relative humidity is shown in eq. 3.8 below.

$$\frac{D_i}{D_{i,ref}} = f_1(T).f_2(t_e).f_3(h) \qquad (3.8)$$

This provides an indication of the permeability of concrete in terms of the ratio of diffusivity of concrete under field conditions, ($D_i$) and reference condition ($D_{i,ref}$).

The principles and concepts discussed above related to the parametric study for the prediction of chloride penetration were applied to four mix designs evaluated in this research project: Lubbock, El Paso, Fort Worth and San Antonio mix designs. The Lubbock mix design was the "old" Lubbock mix that did not use pozzolanic materials for cement replacement and has since been discontinued.

### *3.1.3.1 Lubbock Site*

In the Lubbock mix design, the water/cement (w/c) ratio was 0.44, which is considered moderately high. So, the ratio between the diffusion coefficient for $t_e \rightarrow \propto$ and the diffusion coefficient for $t_e$ at the age of 28 days $\zeta$ can be taken as 0.6 (Saetta et al., 1993) from Table 3.6. For the Lubbock site, correction factors for temperature, relative humidity and age of concrete were calculated for historical weather data obtained from *Lubbock Avalanche Journal* (2000, 2001). The results are presented in Table 3.7 for a relative humidity of 40 percent and for average, high and low temperatures.

From Table 3.7 and RCPT results, it can be predicted that the actual diffusivity of concrete will be much lower in the field condition than found in the laboratory.

**Table 3.7** Temperature Correction Factor at Different Lubbock Temperatures at 40% RH (Using Eq. 3.4, 3.5, 3.6 and 3.7)

| Age (days) | f($t_e$) for $\zeta$=0.6 | Temperature (°F) | | | | | | Combined Factor | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Avg. (°F) | Factor f(T) | High (°F) | Factor f(T) | Low (°F) | Factor f(T) | At Avg. T | At High T | At Low T |
| 56 | 0.88 | 80.5 | 0.033 | 101 | 0.046 | 60 | 0.023 | 0.03 | 0.04 | 0.02 |
| 90 | 0.82 | 81.2 | 0.034 | 100 | 0.045 | 59 | 0.023 | 0.028 | 0.04 | 0.02 |
| 365 | 0.71 | 70.1 | 0.028 | 95 | 0.042 | 44 | 0.017 | 0.02 | 0.03 | 0.012 |

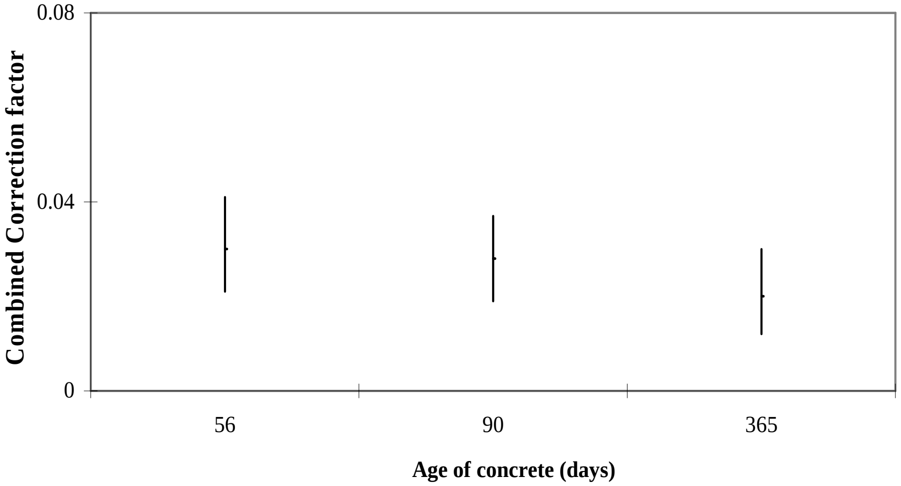


**Figure 3.39** Change in Ratio of Intrinsic Diffusion Coefficient ($D_{field}/D_{ref}$) with Age of Concrete (Lubbock)

The above figure shows the change in overall combined correction factor, or the ratio between diffusion coefficient at field condition and at laboratory with the age of concrete.

### *3.1.3.2 El Paso Site*

Like the Lubbock mix design, w/c of El Paso site was also low (w/c = 0.42). So, for this case the parameter $\zeta$ can also be taken as 0.6 (from Table 3.6). Weather information was collected from the *Lubbock Avalanche Journal*.

**Table 3.8** Correction Factor for Different Variables (Using Eq. 3.4, 3.5, 3.6 and 3.7)

| Age day | f(te) for ζ= 0.6 | h (%) | f(h) | Temperature (°F) | | | | | | Combined Factor | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Avg. (°F) | f(T) | High (°F) | f(T) | Low (°F) | f(T) | At Avg. T | At High T | At Low T |
| 21 | 1.06 | 52 | 0.07 | 81 | 1.14 | 94 | 1.41 | 68 | 0.92 | 0.08 | 0.10 | 0.07 |
| 28 | 1 | 49 | 0.06 | 79 | 1.10 | 92 | 1.37 | 66 | 0.88 | 0.06 | 0.08 | 0.05 |
| 56 | 0.88 | 30 | 0.02 | 82 | 1.16 | 96 | 1.46 | 68 | 0.92 | 0.02 | 0.02 | 0.013 |
| 113 | 0.8 | 32 | 0.02 | 49 | 0.65 | 60 | 0.79 | 38 | 0.53 | 0.01 | 0.01 | 0.007 |
| 365 | 0.71 | 40 | 0.03 | 82 | 1.16 | 95 | 1.43 | 69 | 0.93 | 0.03 | 0.03 | 0.02 |

The ratio of intrinsic diffusion coefficient, $D_{field}/D_{ref}$ *was* found to be the lowest at age of 113 days of curing, because at this age the relative humidity and temperature in the field were low enough to reduce the diffusivity of the concrete.

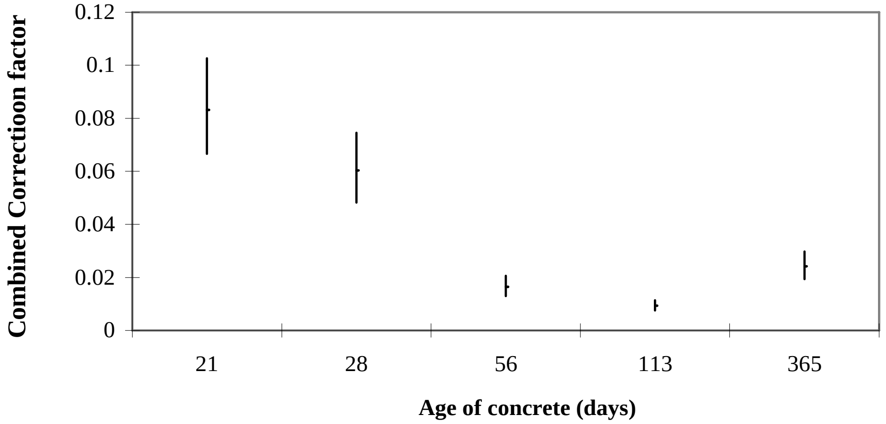


**Figure 3.40** Change in Ratio of Intrinsic Diffusion Coefficient ($D_{field}/D_{ref}$) with Age of Concrete (El Paso)

### *3.1.3.3 Fort Worth Site*

In the Fort Worth mix design, the w/c ratio was 0.36, which is lower than the Lubbock and El Paso sites. So, the ratio between the diffusion coefficient for $t_e \rightarrow \propto$ and the diffusion coefficient for $t_e$ at the age of 28 days $\zeta$ can be taken as 0.8 (From table 3.6). Weather information was collected from the *Dallas Morning News*.

**Table 3.9** Correction Factor for Different Variables (Using Eq. 3.4, 3.5, 3.6 and 3.7)

| Age day | f(te) for ζ= 0.8 | h (%) | f(h) | Temperature (°F) | | | | | | Combined Factor | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Avg. (°F) | f(T) | High (°F) | f(T) | Low (°F) | f(T) | At Avg. T | At High T | At Low T |
| 28 | 1 | 86 | 0.91 | 73.5 | 1.01 | 79 | 1.10 | 68 | 0.92 | 0.92 | 1.01 | 0.83 |
| 56 | 0.94 | 93 | 0.99 | 51.5 | 0.68 | 57 | 0.75 | 46 | 0.61 | 0.64 | 0.70 | 0.57 |
| 110 | 0.9 | 97 | 1 | 37.5 | 0.52 | 39 | 0.54 | 36 | 0.51 | 0.47 | 0.48 | 0.46 |

Though there was cool weather at the age of 28, 56 and 110 days at the Fort Worth site, the relative humidity was high. This is the probable reason for the high value of the ratio of intrinsic diffusion coefficient. If the pore relative humidity remains high, the capillary pores act as an interconnecting channel, which facilitates the diffusion of concrete.

**Figure 3.41** Change in Ratio of Intrinsic Diffusion Coefficient ($D_{field}/D_{ref}$) with Age of Concrete (Fort Worth)

### *3.1.3.4 San Antonio Site*

For San Antonio site, w/c ratio was 0.44. So, for this case the parameter $\zeta$ can be taken as 0.6 (from Table 3.6). Weather information was collected from the *Houston Chronicle*.

**Table 3.10** Correction Factor for Different Climatic Variables at 56 Days

| Temperature (°F) | f(T) | f($t_e$) | Relative Humidity (%) | | | | Combined Factor | |
|---|---|---|---|---|---|---|---|---|
| | | | High | f(h) | Low | f(h) | At high RH | At low RH |
| 87 | 1.26 | 0.883 | 97 | 0.999 | 56 | 0.094 | 1.113 | 0.105 |
| 97 | 1.48 | 0.883 | 97 | 0.999 | 56 | 0.094 | 1.305 | 0.123 |
| 76 | 1.05 | 0.883 | 97 | 0.999 | 56 | 0.094 | 0.93 | 0.088 |

Depending on the relative humidity of the field the correction factor varies. From Table 3.10, it is evident that with increases in pore humidity, permeability of concrete is high.

# CHAPTER IV

# CHLORIDE PONDING TEST PROGRAM AND ANALYSIS OF RESULTS

## 4.1 Test Program

All specimens were tested according to AASHTO T259 and AASHTO T260. These methods determine the resistance of concrete specimens to chloride ion penetration; specifically, the tests are intended to determine the effects of concrete property variations on the concrete's resistance to chloride ion penetration. Variations in the concrete may include, but are not limited to, changes in cement type and content, water cementitious materials ratio, aggregate type and proportions, admixtures, treatments, curing, and consolidation. The test process requires three steps – casting and ponding the samples, drilling the samples, and testing the samples.

### 4.1.1 Casting and Ponding the Specimen

As shown in Table 2.2 on page 14, 18 samples (12''×12'') were cast for each site. As mentioned above, the curing periods studied were 0, 2, 4, 8, 10, and 14 days – three samples for each time period. From these three samples, two were ponded and one was a control (Figure 4.1). For all the laboratory specimens of the test sites, specific codes were assigned (Figure 4.2).

**Figure 4.1** Fort Worth Sample

After the curing phase ended for each specimen, the samples were stored in the drying room until they reached 28 days of age. Immediately after the specified drying period stipulated above (i.e., the 29th day of specimen age), 2.2 ± 1.6 mm of the slab surface from some sites was abraded using grinding or sandblasting techniques because the concrete was to be subjected to wear from vehicle traffic. If the concrete or treatment was to be used on surfaces not subject to wear from vehicle traffic (which is not the case in this research project), then the abrading step may be omitted. Figures 4.2 and 4.3 show two abraded samples, one with grinding and the other with sandblasting. In our test, the surfaces of New Lubbock and Fort Worth were not abraded, while for the other sites, 50% of the surface was abraded using the sandblasting technique.

**Figure 4.2** Sample Abraded Using Grinding Technique

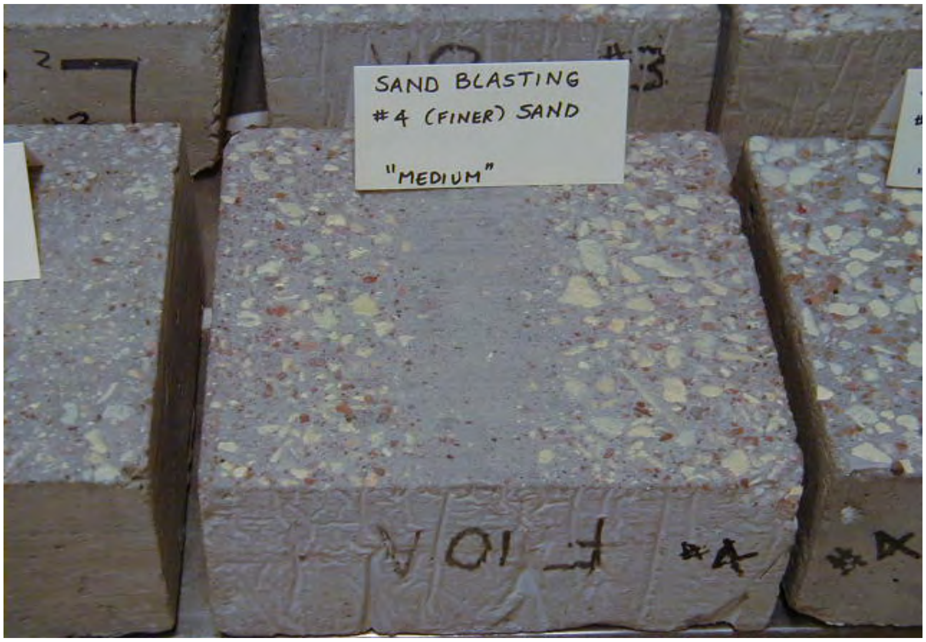


**Figure 4.3** Sample Abraded Using Sandblasting Technique

After that, approximately 19 mm high by 13 mm wide dams were placed around the top edge of all specimens except those that became the control specimen (Figure 4.4). All slabs were returned to the drying room as specified for an additional 13 days (i.e., until 42 days of age).

**Figure 4.4** Construction of Dams for the Houston Samples

The slabs (or specimens) with dams and the 6-inch core field specimen were subjected to continuous ponding with 3% sodium chloride solution to a depth of approximately 13 mm (0.5”) for 90 days for Fort Worth and New Lubbock, while the other sites were ponded for 180 days. The solution was prepared by diluting crystals of sodium chloride in a defined amount of distilled water. Glass plates were placed over the ponded solutions to retard evaporation of the solution (Figure 4.5). Placement of the glass plates was done in such a manner that the surface of the slab was not sealed from the surrounding atmosphere. Additional solution was added if necessary to maintain the 13 mm (0.5”) depth. All slabs remained in the drying room for the duration of the ponding period. A summary of the information on all the sites is presented in Table 4.1.

After 90 days of exposure, unless stated elsewhere, the solution was removed to dry and then the surfaces were wire brushed until all salt crystal buildup was completely removed.

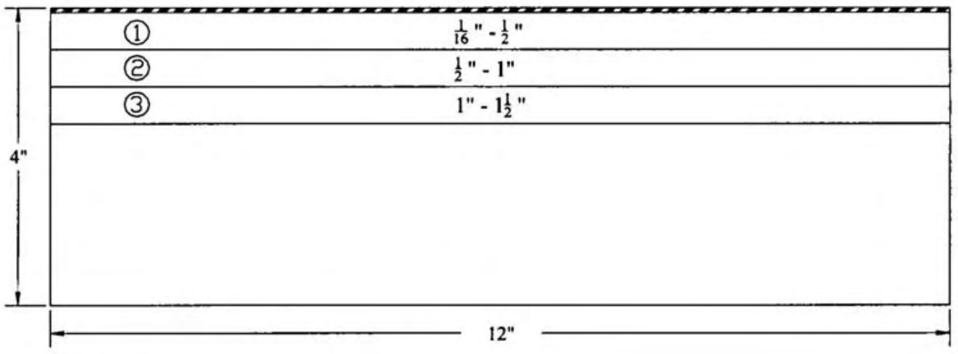
**Figure 4.5** Houston Samples Ponded for 180 Days

**Table 4.1** Test Program Setup

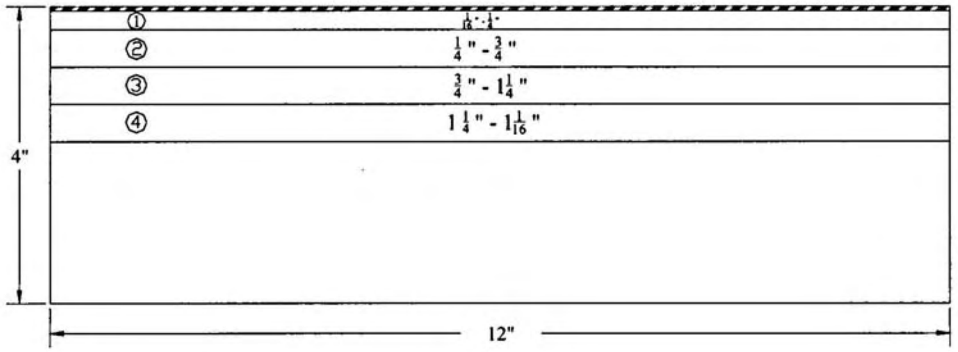
|  | **Districts** |  |  |  |  |  |  |
|---|---|---|---|---|---|---|---|
| Parameter | San Antonio | El Paso | Pharr | Fort Worth | New Lubbock | Atlanta | Houston |
| Casting | Field | Field | Lab | Field | Lab | Lab | Field |
| Abraded | 50% | 50% | 50% | No | No | 50% | 50% |
| # 12”×12” | 18 | 18 | 18 | 18 | 18 | 18 | 18 |
| Ponding Duration | 180 days | 180 days | 180 days | 90 days | 90 days | 180 days | 180 days |
| # of drill depth | 4 | 4 | 4 | 4 | 4 | 4 | 4 |
| # of drilling | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| # of tests | 144 | 144 | 144 | 144 | 144 | 144 | 144 |
| Total # of tests | 1008 |  |  |  |  |  |  |
| Drill pattern (see p. 92) | B | B | B | B | B | B | B |

### 4.1.2 Drilling the Specimen

AASHTO T259 recommends that the samples should be obtained from each specimen at each of the following depths as seen in Pattern A in Figure 4.6. However, because of the different setup of the drilling machine that was used and in order to get more refined information at the top surface of the specimen, Pattern B in Figure 4.6 was adopted.



Drill Pattern A



Drill Pattern B

**Figure 4.6** Drill Patterns A and B

The Gilson Model HM-343 (Figure 4.7) was used for the drilling procedure. This machine is designed to take precise concrete samples for chloride ion penetration evaluation, and it has the following unique features:

1) No cross contamination among samples. A clean sample path is created when each sample is taken.
2) Rapid sample collection. The collection vessel, filter, and tube are a single unit. The sample container and tube are inserted into a fixed position ready to collect the sample. The collection containers may be discarded after use.
3) Only two drill bits are needed. A 5/8” diameter bit drills a pilot/connected companion hole for the powder sample collection system.

4) The heavy gauge drill guide and template ensure vertical holes and correct overlap of the sample and companion powder collecting holes.
5) A powerful vacuum motor removes the sample powder produced as the core bit progresses.

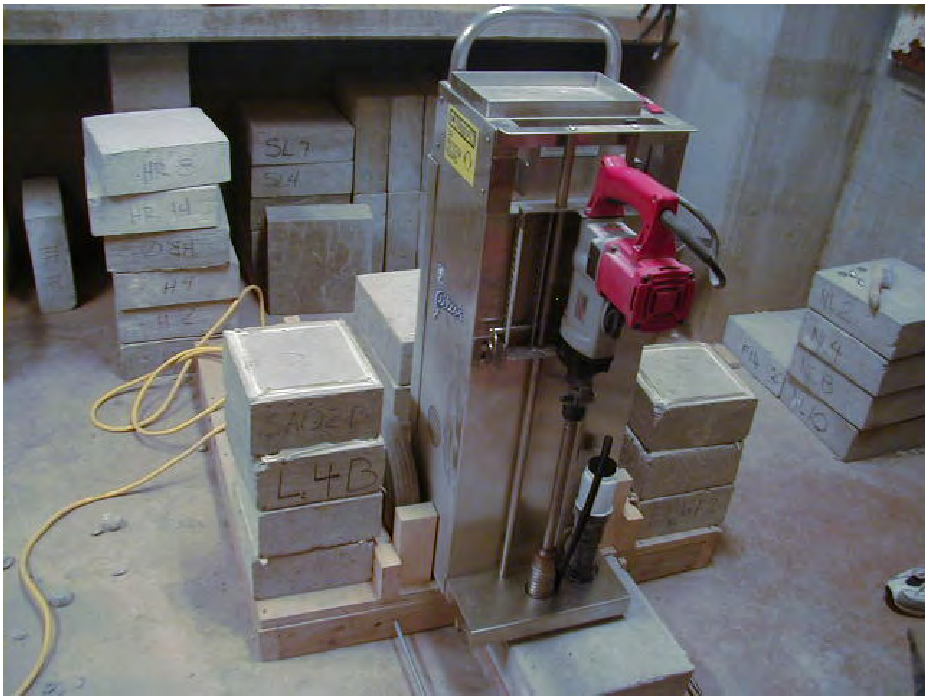


**Figure 4.7** Gilson Model HM-343

The sample drilling assembly’s components are:

1) A two-wheeled self contained cart with stainless steel panels and service tray. All system components are contained in the unit.
2) Heavy duty rotary impacts hammer drill.
3) A hole drilling template.
4) A drilling guide including ½” stop controls.
5) A vacuum motor and filter.
6) A sampling assembly including vacuum piping, tubing guide, and sample bottle holder.
7) Single-use sample collection bottles, bags, and tubes.
8) Masonry bits: A 2” core bit and a 5/8” companion hole bit.

The steps of drilling are illustrated in Figure 4.8 and are as follows:

1) Identify/mark the location of holes to be drilled. Two holes per sample are typically created.
2) Attach the 5/8” drill bit to the drill motor and position it over the mark with power switch off.
3) Turn on the “Main/Aux” power switch. Then turn on the “Vacuum/Drill” switch and drill to the desired depth. The average maximum drilling depth is approximately 4”. The drill stop can be used to control drilling depth by moving the clevis pin on the depth stop gauge to the desired stop point.
4) Turn the drill off when the stop point is reached. One of the advantages of model HM-343 is that it allows the bit to advance ¼” beyond the drill stop set point if the operator continues drilling after the stop point is reached. An additional safety item is that the drill will not operate unless the vacuum switch is “On”. To reduce time, drill several companion holes before changing to the core bit.
5) Remove all powder and debris from the companion hole and core hole area after the companion hole has been drilled. A sample collection assembly is used to collect the powder from the companion hole.
6) Remove the companion hole and install the core bit.
7) Align the cart’s drilling template over the drilled and cleaned companion hole. A sample collection bottle with collection bag and tube is placed into the vacuum holder. The collection tube is guided into the companion hole and located at the end of the tube near the top of the companion hole. After the coring bit has advanced about an inch, the tube is pushed to the middle of the companion hole.
8) Stop the first drill guide at ¼” depth. All subsequent stop points are at ½” intervals. So ¼” is drilled into the concrete sample, the material is collected into the bag, and the drill guide is reset to the first ½” collection depth set point.
9) Load the new sample collection bottle using the same procedure described above. When the drill stop pin is set, drilling to ½” depth is completed. The procedure is repeated for the next ½” and ¾” to 1¼” intervals. Thus, for each hole, three powder samples are collected if the ¼” is discarded or four powder samples if the ¼” is collected.
10) Mark each sample collection bottle with four numbers:

- Sample number
- Site name
- Hole number
- Depth number

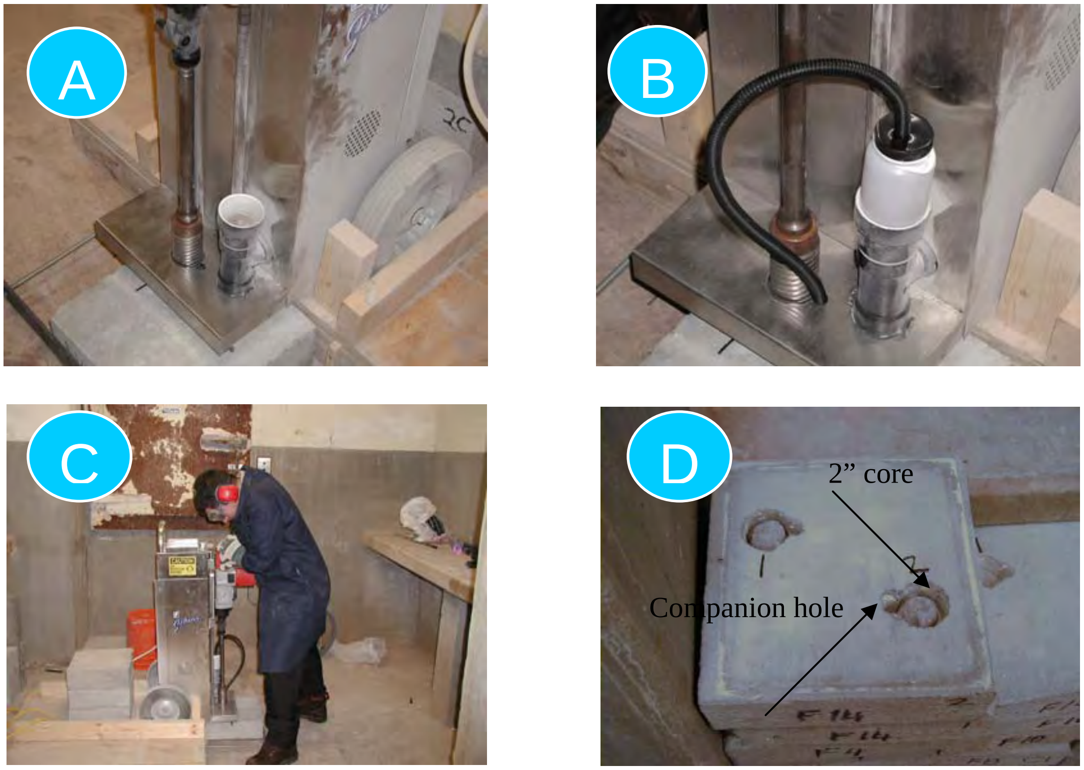


**Figure 4.8** Drilling Steps

4.1.3 Testing the Specimen

After the drilling phase was complete, the sample collection bottles were moved to a partially controlled environmental lab (i.e., temp 65° ± 10°), where the samples were tested to determine the chloride ion in the concrete. At the beginning of this research project, the AASHTO T260 guidelines were followed explicitly.

The AASHTO T260 method covers procedures to determine the acid soluble chloride ion content or the water soluble chloride ion content of aggregates, Portland cement, mortar, and concrete. The total amount of chloride is usually equal to the acid soluble chloride. However, organic additives or minerals that contain acid insoluble chloride may be present in concrete and concrete raw materials. These constituents may become acid soluble during long-term exposure to the alkaline environment in concrete or mortar.

The age of concrete mortar or hydrated Portland cement at the time of sampling has an effect on the water soluble chloride ion content.

Two procedures are presented in AASHTO T260. The first is related to the determination of acid-soluble chloride ion and the second is related to the determination of the water-soluble chloride ion content. In this project test, the second procedure was followed. This procedure is described as follows:

1) Initially, the mass to the nearest milligram of a 3 g powdered sample representative of the material under test was determined. In general, the concrete powder was received in a pulverized condition. However, when samples were received in the laboratory in other than pulverized condition, the sample was crushed and ground to a powder. All crushing was done dry (i.e., without water). All material passed through a 0.300 mm ($N_0$.50) sieve. All pulverizing tools and sieve were washed with alcohol or distilled water and dried before use with each separate sample.
2) The sample was transferred quantitatively to a beaker of capacity 150 to 200 mL. Sixty to seventy mL of distilled water was added. The beaker was covered with a watch glass and brought to a boil on a hot plate magnetic stirrer using a small magnet. The sample was boiled for 5 minutes and then left standing for 24 hours in a fume-free atmosphere (Figure 4.9).
3) The clear liquid was filtered in the beaker through double filter paper (Whatman No.41 over No.40 or equivalent) into a 250 mL beaker (Figure 4.10). Care must be taken to quantitatively transfer any adherent drops on the watch glass. A stirring rod generally was used to aid in the transfer. Sufficient hot distilled $H_2O$ was added to cover any residue left in the original beaker. The solution was stirred for 1 minute on a magnetic stirrer and filtered into a 250 mL beaker with a swirling action. The beaker and the stirring rod were washed once into the filter using hot distilled $H_2O$. The filter paper was lifted carefully from the funnel, and the outside surface of the paper was washed with hot distilled $H_2O$. The paper was set aside, and the interior of the funnel and its tip were washed with hot

distilled $H_2O$. One to two drops of methyl orange indicator was added to the 150 mL beaker. Next, concentrated $HNO_3$ was added one drop at a time (four to seven drops) with continuous stirring until a permanent pink to red color was obtained. If the sample contained blast furnace slag or other sulfide-bearing material, 3 mL of hydrogen pyroxide (30% solution) was added. Once all of the above was completed, the volume of the solution was brought to 125 to 150 mL using distilled $H_2O$.

4) The $Cl^-$ electrode was filled with the solution recommended by the manufacturer (Thermo Orion 2003). The solution was read with a millivolt gauge to determine the approximate equivalence point by immersing the electrode in a beaker of distilled $H_2O$. The approximate millivolt reading was unsteady in $H_2O$ as water contains no ions. Four mL of 0.01 normality NaCl was added to the cooled sample beaker using constant swirling. The beaker of distilled $H_2O$ was removed from the electrode. The electrode was then wiped with absorbent paper and immersed in the sample solution. The entire beaker electrode assembly was placed on a magnetic stirrer and gentle stirring began. Using a calibrated buret, standard 0.01 normality $AgNO_3$ solution gradually was added to bring the millivolt reading to -40 mV of the equivalence point determined in distilled $H_2O$. The amount of $AgNO_3$ required was recorded. Next, standard 0.01 normality $AgNO_3$ solution was added in 0.10 mL increments and the millivoltmeter reading was recorded after each addition.
5) As the equivalence point was approached, equal additions of $AgNO_3$ solution caused large changes in the millivoltmeter reading. Past the equivalence point, the changes per unit volume would again decrease. The titration was continued until the millivoltmeter reading was at least 40 mV past the approximate equivalence point. The endpoint of the titration was usually near the approximate equivalence point in distilled water and may be determined by two methods:
   a) By calculating the differences in millivoltmeter readings between successive $AgNO_3$ additions (Fig 4.11). The endpoint is the peak of the plot.
   b) By plotting the volume of $AgNO_3$ solution added versus the millivoltmeter readings, the endpoint will correspond to the point of inflection of the curve as indicator (Fig 4.12).

The endpoint will be near the midpoint, which produces the largest change in millivolt-meter readings. It may be determined by plotting midpoints versus differences and defining the $AgNO_3$ volume, which corresponds to the maximum difference on a smooth symmetrical curve drawn through the points. The percent $Cl^-$ is calculated from the following equation (AASHTO T260):

$$Cl^- \, percent = \frac{(3.5443(V_1 N_1 - V_2 N_2))}{W}, \qquad (4.1)$$

where $V_1$ is the endpoint in mL of $AgNO_3$, $V_2$ is the normality of $AgNO_3$, W is the mass of original concrete sample in grams, $V_2$ is the volume of NaCl solution added in mL, and $N_2$ is the normality of NaCl solution.

**Figure 4.9** Boiling the Specimen for Five Minutes

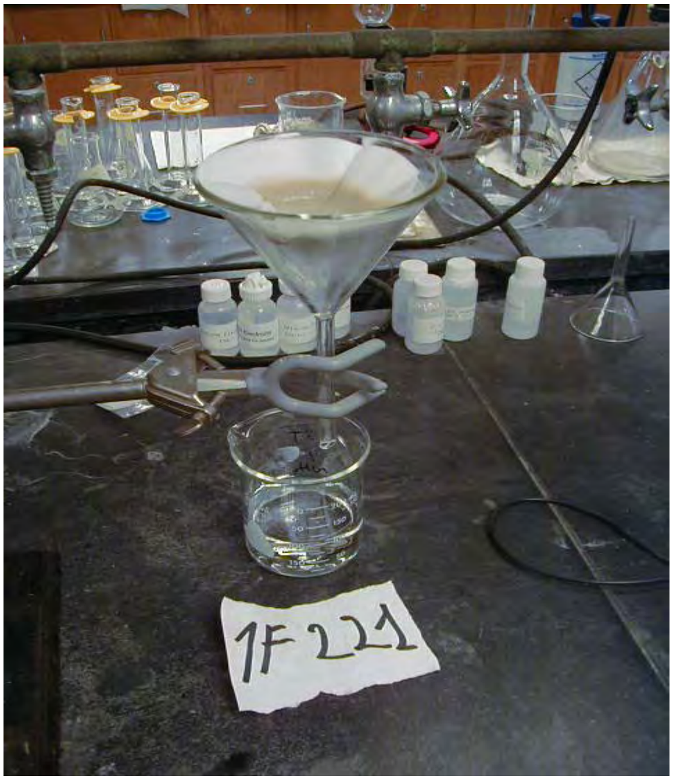


**Figure 4.10** Filtration of the Solution

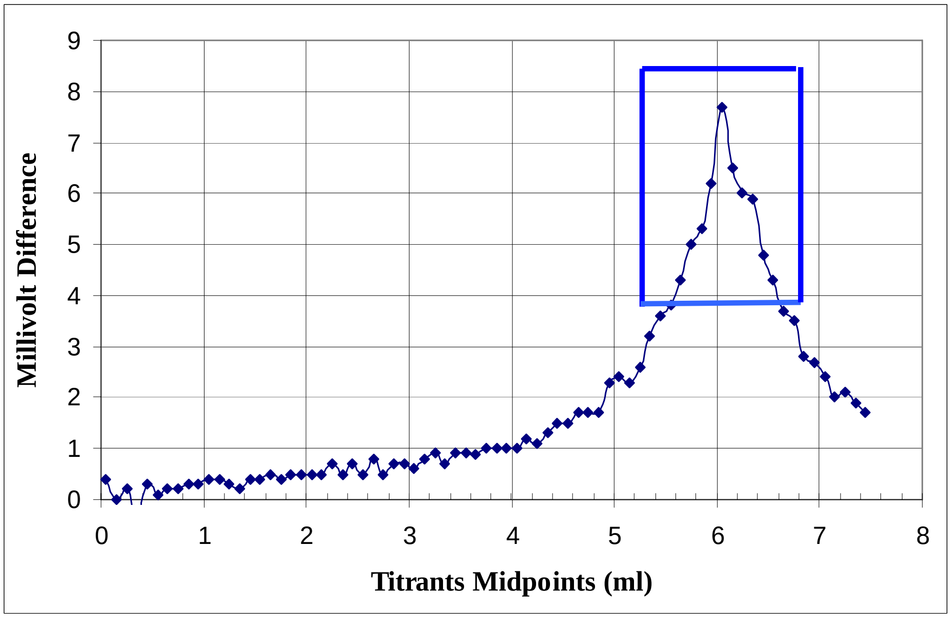


**Figure 4.11** Max Millivolt Difference Corresponding to the Endpoint

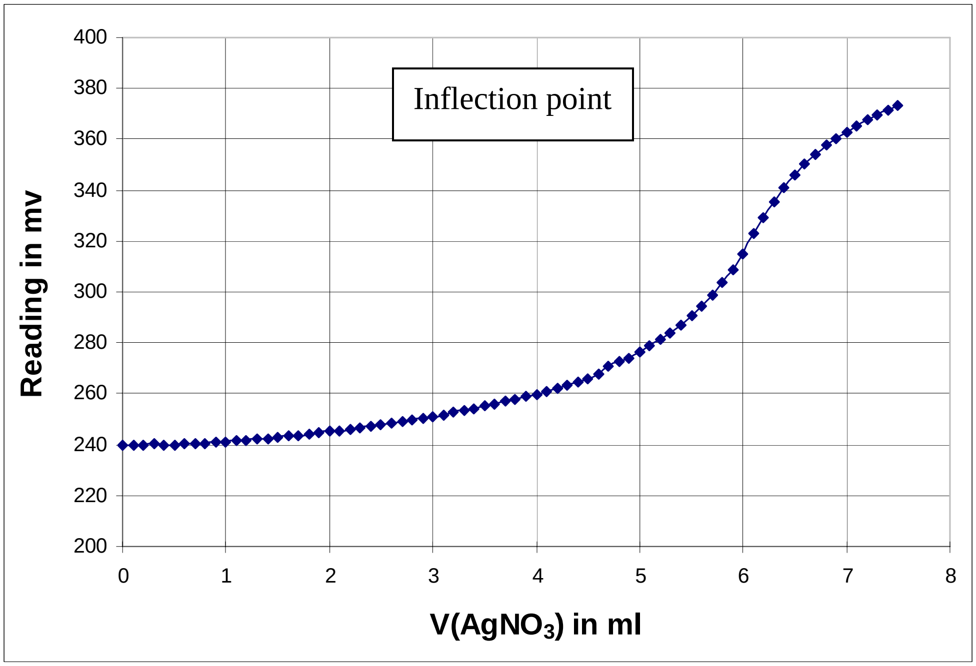


**Figure 4.12** Point of Inflection Corresponds to the Endpoint

## 4.2 Analysis of Ponding Test Results

These experiments were conducted to investigate the effect of wet-mat curing duration on chloride ion permeability in the ponding test. For each sample, two holes were drilled on the unabraded finished surfaces of the specimens from Fort Worth and New Lubbock districts and on the abraded finished surfaces for Houston, San Antonio, El Paso, Pharr, and Atlanta (see Table 2.2). Samples were collected from four different depths (Figure 3.8): 1/16” to 0.25”, 0.25” to 0.75”, 0.75” to 1.25”, and 1.25” to 1.75” for a total of 144 samples per site (i.e., 2 holes × 4 depths × 18 specimen per site). The chloride content for each sample was determined using an electrode calibrated with standard solutions.

### 4.2.1 Fort Worth Site

For the Fort Worth site, concrete was mixed at a water cementitious ratio of 0.43, and 25% of the required cement content was replaced with class F fly ash. The cement: fly ash: fine aggregate: coarse aggregate ratio was 1.00: 0.33: 3.15: 4.66 (volumetric). The coarse aggregate used was limestone. The test results for Fort Worth are shown in Table 4.2. Included in this table are the chloride concentrations for all the samples. It should be noted that each number in the table is an average of four numbers. The first, second, third, and fourth columns show the chloride concentration for the first, second, third, and fourth layers respectively. As can be seen in Table 4.2, within each column, as the curing duration increases, the chloride concentration decreases, thus concrete becomes less permeable. For example, the chloride concentration decreases from 57 ppm (part per million) for FA (0 day curing) to 44 ppm for FF (14 days curing), and this is due to the fact that the concrete microstructure has been changed due to the curing duration. In fact, concrete microstructure becomes denser (i.e., less permeable) as the curing duration and sample depth increase.

**Table 4.2** Concentrations for Fort Worth Ponding Specimens in ppm

| **Non Abraded – Ponded (90 days) – Type I/II Cement – 22% FA (class F) - LM** | | | | |
|---|---|---|---|---|
| | 0-1/4” | ¼”-3/4” | ¾-1 ¼” | 1 ¼”-1 ¾” |
| **FA (0 days)** | 57 | 44.5 | 7.5 | 2.5 |
| **FB (2 days)** | 51 | 10.75 | 7 | 3 |
| **FC (4 days)** | 48 | 31.5 | 5.5 | 2.5 |
| **FD (8 days)** | 47 | 18.5 | 3.75 | 2.75 |
| **FE (10 days)** | 44 | 18 | 3.75 | 2.25 |
| **FF (14 days)** | 44 | 16 | 2.75 | 2.75 |

To further illustrate the effect of curing on the permeability of concrete, Figures 4.13 and 4.14 are shown. These figures represent a set of relations between the chloride concentration and the curing duration for the four depths under study. As shown in these figures, chloride was concentrated in the upper two layers (up to 0.75”), whereas the chloride content was extremely small in the bottom two layers and almost negligible in the bottom layer. This extreme decline in chloride concentration through the depth of the specimen is important to observe. As the concrete samples were dried for 28 days prior to ponding, when they came in contact with the salt solution, the transport of the solution due to capillary action prevailed only in the superficial layers that present open and connected pores. Even if the pores were connected into the deep interior of concrete, the capillary transport takes place only to a certain distance (e.g., 1.25” in the case of the Fort Worth site) as the pore walls provide resistance to the flow of liquid. In general, the degree of this resistance depends on pore dimensions, with smaller pores presenting larger resistances than larger pores.

Another important conclusion shown in Figure 4.13 is that the effect of curing on the chloride concentration after 8 days was extremely small. Specifically, while the chloride concentration decreased from 44.5 ppm (0 days curing, layer 1) to 18.5 ppm (8 days curing, layer 1), it decreased by only 2.5 ppm from 18.5 ppm (8 days curing, second layer) to 16 ppm (14 days curing, second layer). This is possibly due to the fact that during the first few days of curing, the top layer of the concrete became completely saturated, and the capillary pores in the top layers were filled with water so that even with an interruption of the wet-mat curing, the inner concrete surface would still be saturated with a relative humidity above 80%.

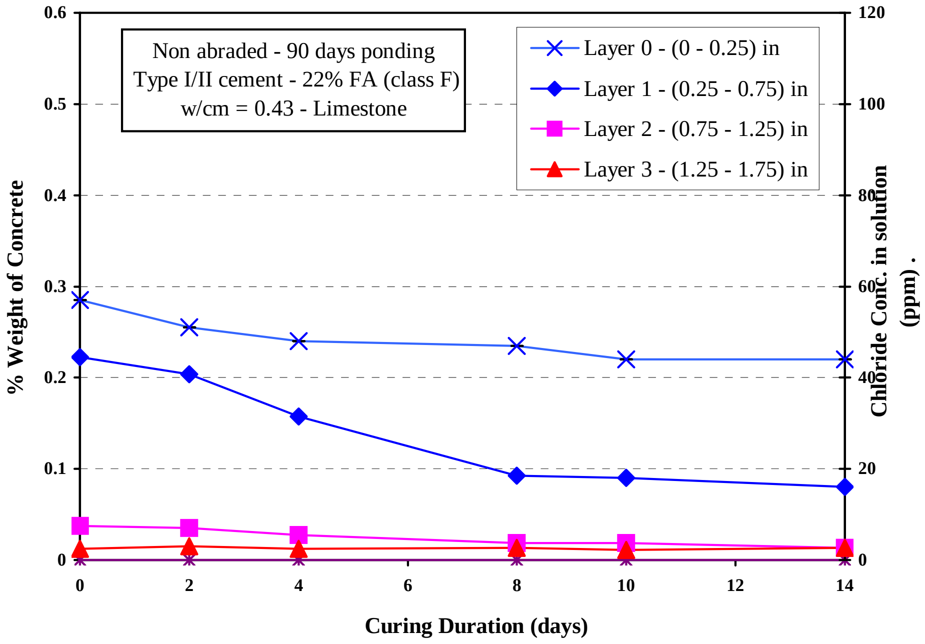


**Figure 4.13** Chloride Concentrations versus Curing Duration for Fort Worth

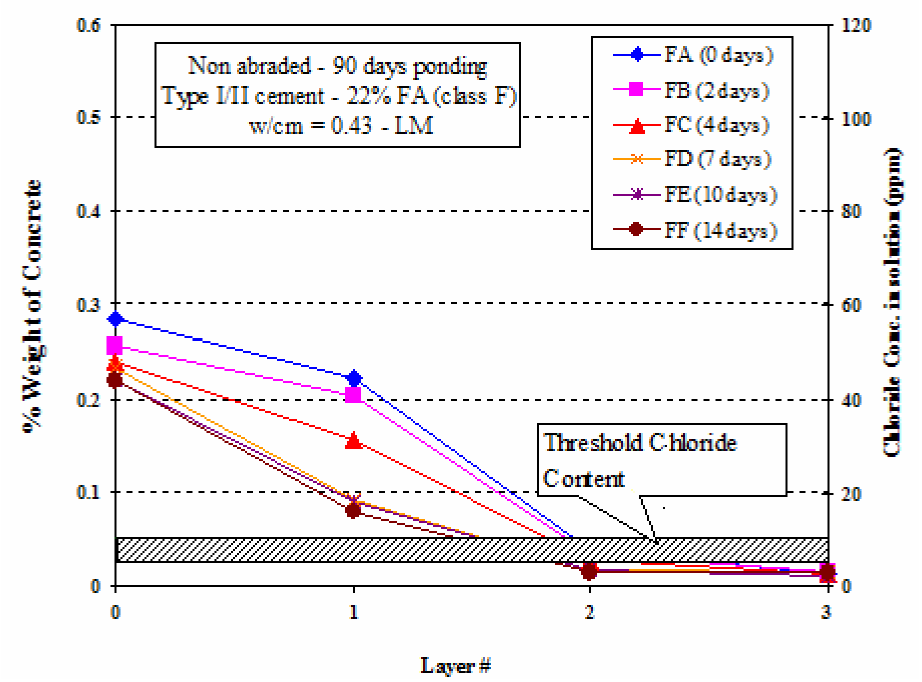


**Figure 4.14** Chloride Concentrations versus Layer Number for Fort Worth

The threshold chloride content identified in Figures 4.14, 4.17, 4.20, 4.23, 4.26, 4.29 and 4.32 were obtained from work conducted by Clear (1983).

Figure 4.15 shows the chloride concentration of all the control samples for the Fort Worth site for all depths. The numbers fall between 2 and 3 ppm for all the samples, which is negligible. In fact, this percent of chloride may have entered the concrete during mixing, either in the water or through the cement ingredients.

Comparing the ponded and control specimens for each curing duration (Figures 4.14 versus 4.15) shows conclusively that the chloride due to ponding was concentrated in the top two layers (up to 0.75”) and partially penetrated the third layer (up to 1.25”). However, the fourth layer remained unaffected as both curves (e.g., ponded and control samples) for all curing durations merged together at layer 3, which extends from a depth between 1.25” and 1.75”. Generally, the location of reinforcement was below this area since all numbers fall below the threshold value of corrosion, which is 0.9 lb/yd$^3$ (equivalent to 5 ppm of chloride in solution).

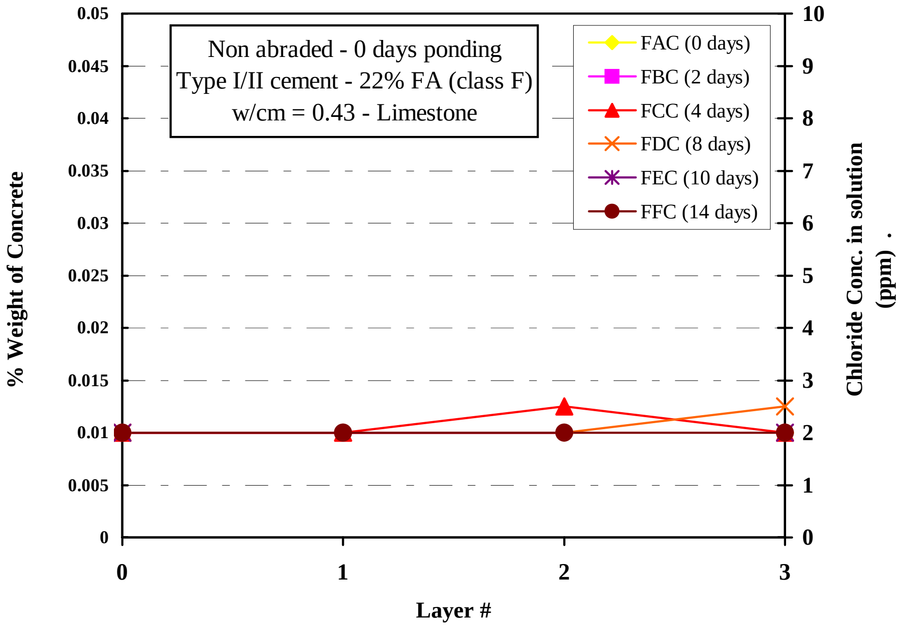


**Figure 4.15** Chloride Concentrations for Fort Worth Control Specimens

### 4.2.2 New Lubbock Site

For the New Lubbock site, concrete was mixed with water cementitious ratio of 0.45, and 31% cement was replaced with class F fly ash. The cement: fly ash: fine aggregate: coarse aggregate ratio was 1.00: 0.54: 3.43: 5.39 (volumetric). The coarse aggregate used was limestone. Table 4.3 shows the test results for New Lubbock. It can be seen from this table that the permeability of concrete is inversely proportional to the curing duration. As the chloride concentration decreased, permeability decreased and vice versa. That is, the chloride concentration decreased from 40 ppm for NLA (0 day curing) to 31 ppm for NLF (14 days curing) and this may be due to the fact that the concrete pore structure had been changed.

**Table 4.3** Concentrations for New Lubbock Ponding Specimens in ppm

| **Non Abraded – Ponded (90 days) – Type I/II Cement – 31% FA (class F) - LM** | | | | |
|---|---|---|---|---|
| | 0-1/4” | ¼”-3/4” | ¾-1 ¼” | 1 ¼”-1 ¾” |
| **NLA (0 days)** | 40 | 25.5 | 4.5 | 3 |
| **NLB (2 days)** | 35 | 24 | 4.5 | 3.5 |
| **NLC (4 days)** | 31 | 17 | 4.25 | 3 |
| **NLD (8 days)** | 31 | 16.2 | 3.75 | 3 |
| **NLE (10 days)** | 30.5 | 16 | 3.75 | 3 |
| **NLF (14 days)** | 31 | 12.75 | 3.25 | 2.25 |

The importance of curing on permeability of concrete is illustrated in Figures 4.16 and 4.17. Again, chloride was concentrated in the top two layers (up to 0.75”), whereas the chloride content was negligible in the bottom two layers. This extreme decline in chloride concentration through the depth of the specimen may be due to the resistance of the pore walls to the flow of liquid. As shown in Figure 4.16, the effect of curing on layers 2 and 3 is almost negligible. This can be explained because moisture movement usually affects only the outer zone of concrete – typically the first inch (Neville 1996). In reinforced concrete, this depth represents all or most of the clear cover. Thus, concrete in the interior of a structural member generally is unaffected by significantly long curing durations. However, the properties of concrete in the outer zone are greatly influenced by curing, and this is well illustrated in Figure 4.16, layer 0. In fact, it is this zone that is subject to weathering, carbonation, and abrasion, and the permeability of the outer zone concrete has a paramount influence on the protection of steel reinforcement from corrosion (Neville 1996).

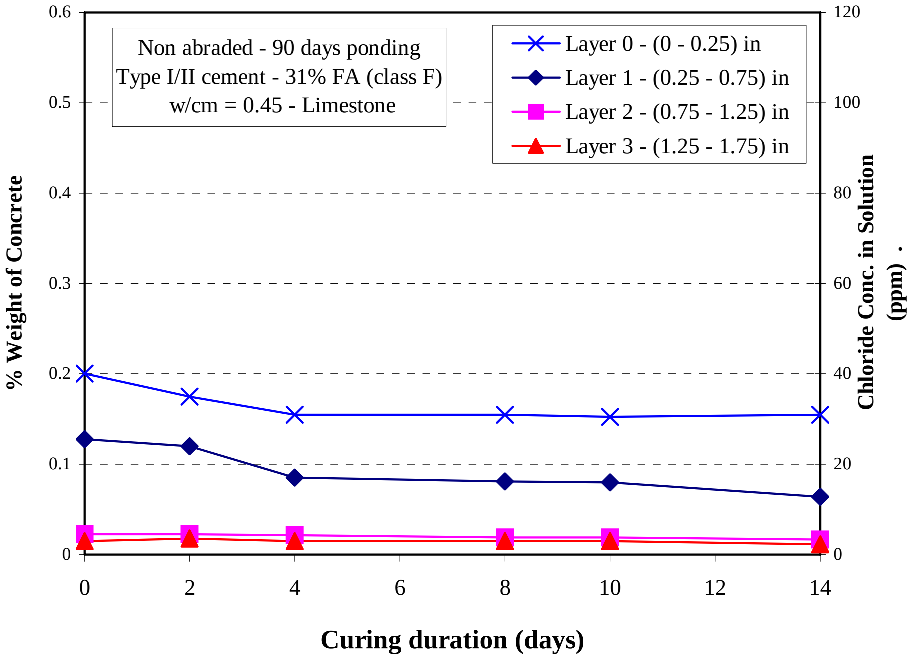


**Figure 4.16** Chloride Concentrations versus Curing Duration for New Lubbock

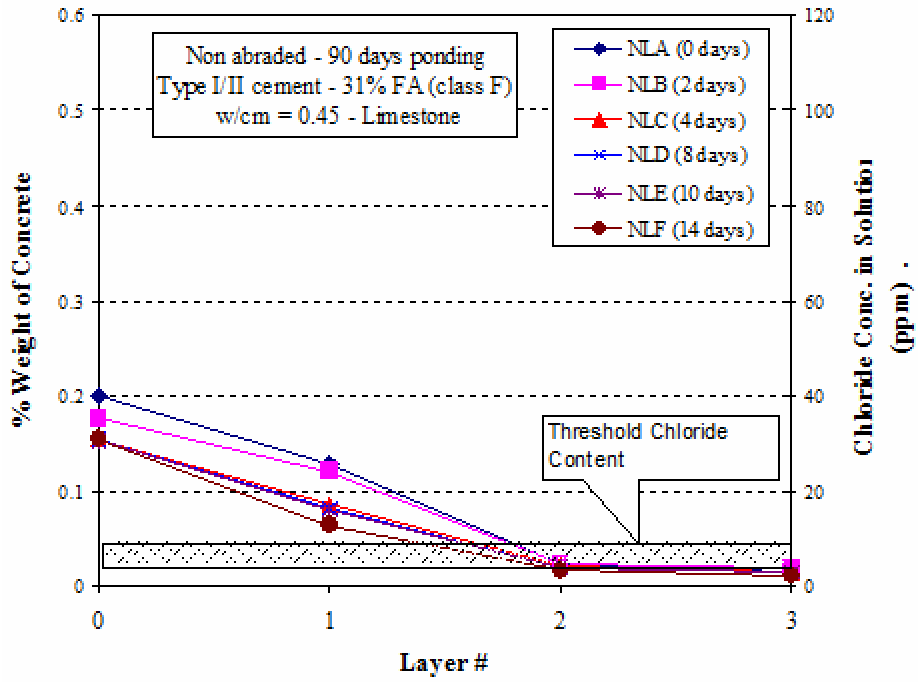


**Figure 4.17** Chloride Concentrations versus Layer Number for New Lubbock

Another important conclusion shown in Figure 4.16 is that the effect of curing on the chloride concentration after 4 days of curing was extremely small. That is, for the New Lubbock mixture, while the chloride concentration decreased from 25.5 ppm (0 days curing, layer 1) to 17 ppm (4 days curing, layer 1), it decreased by only 2.5 ppm from 17 ppm (4 days curing, layer 1) to 12.75 ppm (14 days curing, layer 1). Thus no appreciable improvement (decrease in chloride concentration) was found beyond 4 days of wet-mat curing.

Figure 4.17 shows that the location of steel reinforcement seems to be safe at 1.25” below the top surface since all values of layer 3 fall below the threshold content of 1 to 2 lb/yd$^3$.

The results of control samples for New Lubbock are shown in Figure 4.18. As expected, the chloride content is almost negligible in all layers at different curing regimes. In general, chloride may be incorporated in the concrete through the use of contaminated aggregate or admixtures containing chlorides. In addition, the figure shows that the distribution of chloride in the control specimen within the hardened cement paste was not uniform.

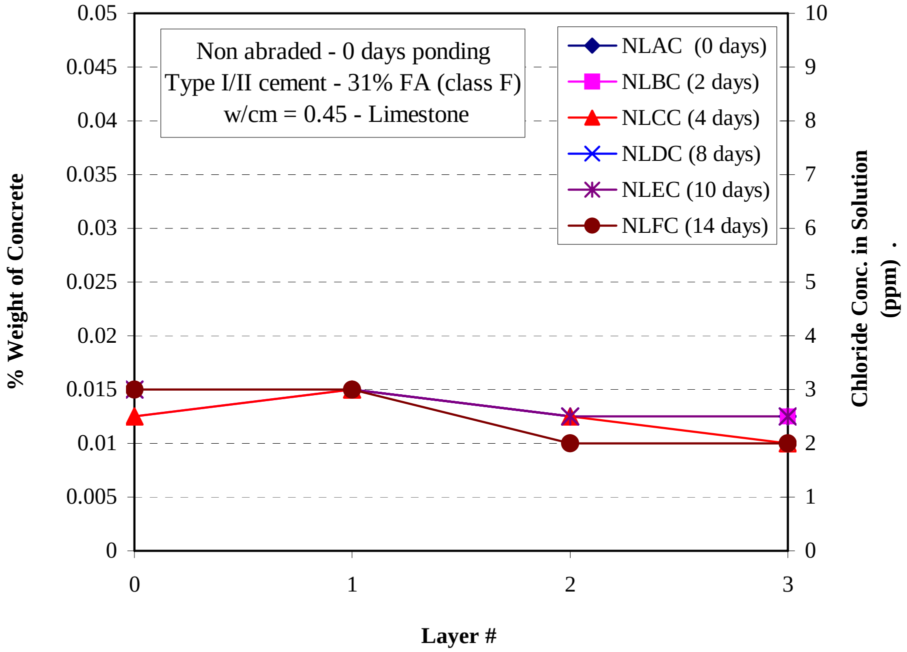


**Figure 4.18** Chloride Concentrations for New Lubbock Control Specimens

### 4.2.3 El Paso Site

For the El Paso site, concrete was mixed at a water cementitious ratio of 0.42, and 50% of cement replaced with ground granulated blast furnace slag (GGBS). The cement: slag: fine aggregate: coarse aggregate ratio was 1.00: 1.00: 4.87: 6.55 (volumetric). The coarse aggregate used was limestone. The chloride permeability of concrete from this site was tested after 180 days of ponding. Table 4.4 shows the results of the El Paso site tests.

It has been found that by using pozzolanic materials (e.g. GGBS), the permeability of concrete reduces to a large extent in the first few days. In this case, the concentration of chloride decreased from 79 ppm to 60.5 ppm (25% decrease over 2 days), while it decreased only by 1.5 ppm over the next 12 days of curing (7.8% decrease).

**Table 4.4** Concentrations for El Paso Ponding Specimens in ppm

| **Abraded –180 days ponding – Type I/II Cement – 50% slag - Limestone** | | | | |
|---|---|---|---|---|
| | 0-1/4” | ¼”-3/4” | ¾”-1 ¼” | 1 ¼”-1 ¾” |
| **EPA (0 days)** | 78 | 41 | 8.5 | 5.75 |
| **EPB (2 days)** | 60.5 | 29.5 | 7.5 | 5.25 |
| **EPC (4 days)** | 59 | 28.5 | 7.25 | 6.75 |
| **EPD (8 days)** | 58.75 | 28 | 9.5 | 7.25 |
| **EPE (10 days)** | 59 | 28 | 9 | 7 |
| **EPF (14 days)** | 59 | 29.5 | 6.75 | 6 |

Figures 4.19 and 4.20 show that the chloride concentration for 0 cure day concrete was much higher than any other concrete for other curing durations. This observation is explained if one concludes that the resistivity of chloride ion penetration grew more in cases of longer curing durations. Also, nearly horizontal current plots were observed after 2 days of curing for all depths, which led to the conclusion that 2 days of curing was sufficient for the El Paso mixture – a mixture with a 50% replacement of cement with ggbs. In general, since a blend of Portland cement and ggbs contains more silica and less lime than Portland cement alone, hydration of the blended cement produce more C-S-H and less lime than Portland cement alone (Neville 1996). The resulting microstructure of the hydrated cement paste is dense. However, the initial

hydration of ggbs is very slow as it depends on the breakdown of the glass by the hydroxyl ions released during the hydration of the Portland cement (Neville 1996). In a manner similar to blended cement containing pozzolans, reaction of ggbs with calcium hydroxide takes place. The progressive release of alkalis by the ggbs, together with the formation of calcium hydroxide by Portland cement, results in a continuing reaction of ggbs over a long period. In the case of the El Paso site, 2 days appeared to be sufficient. However, prolonged moist curing (28 days and beyond) of concrete containing ggbs is particularly important because the initial low rate of hydration results in a system of capillary pores that allows the loss of water under drying conditions. If this loss happens, hydration cannot take place and no improvement in concrete permeability is achieved beyond that. The resistance of the pore walls leads to the concentration of chloride in the top layers and prevents the chloride ions from penetrating deeper into the concrete.

Figure 4.21 shows the chloride concentration for all depths for all of the control samples for the El Paso site tests. All of the values fell between 5 and 6 ppm for all the samples, which is more than expected. In fact, this percent of chloride may have entered the concrete during mixing, either in the water or through the cement ingredients.

To determine the safest place to locate the reinforcement, a comparison was made between the ponded and the control specimens for each curing duration (Figures 4.20 versus 4.21). The top two layers were not the appropriate location since all numbers for all curing durations were much higher than the threshold content, which suggests that the steel reinforcement be placed below layer 3, which extends from 1.25” to 1.75”. Chloride ion values in this area fall below the threshold value of corrosion, which is 1 to 2 lb/yd$^3$ (equivalent to 5 to 10 ppm of chloride in solution).

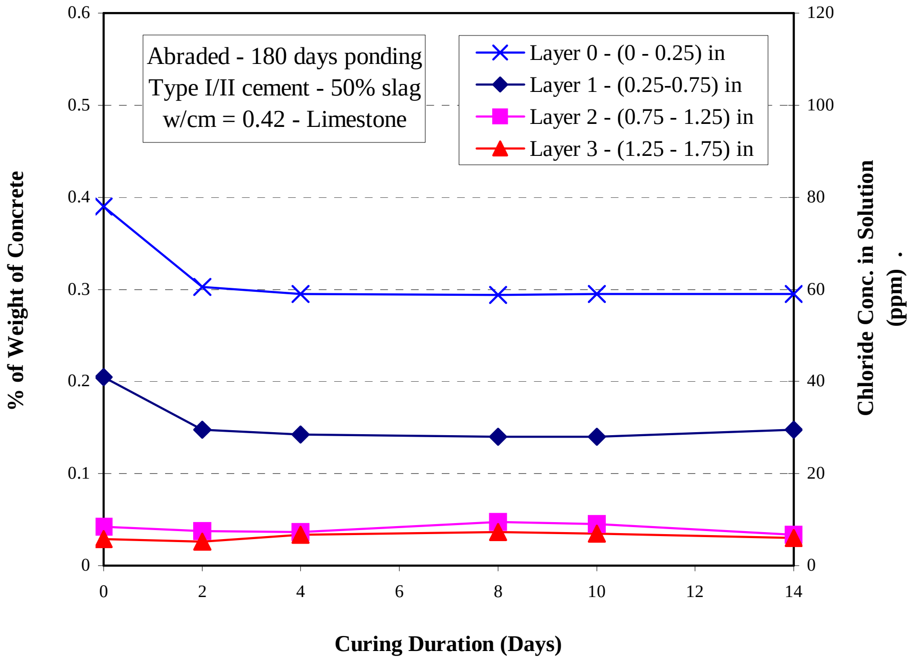


**Figure 4.19** Chloride Concentrations versus Curing Duration for El Paso

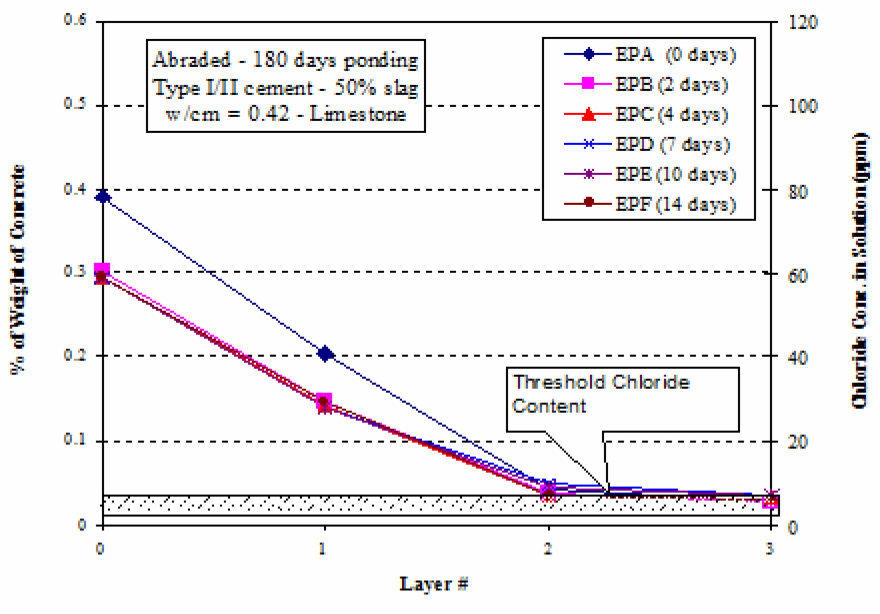


**Figure 4.20** Chloride Concentrations versus Layer Number for El Paso

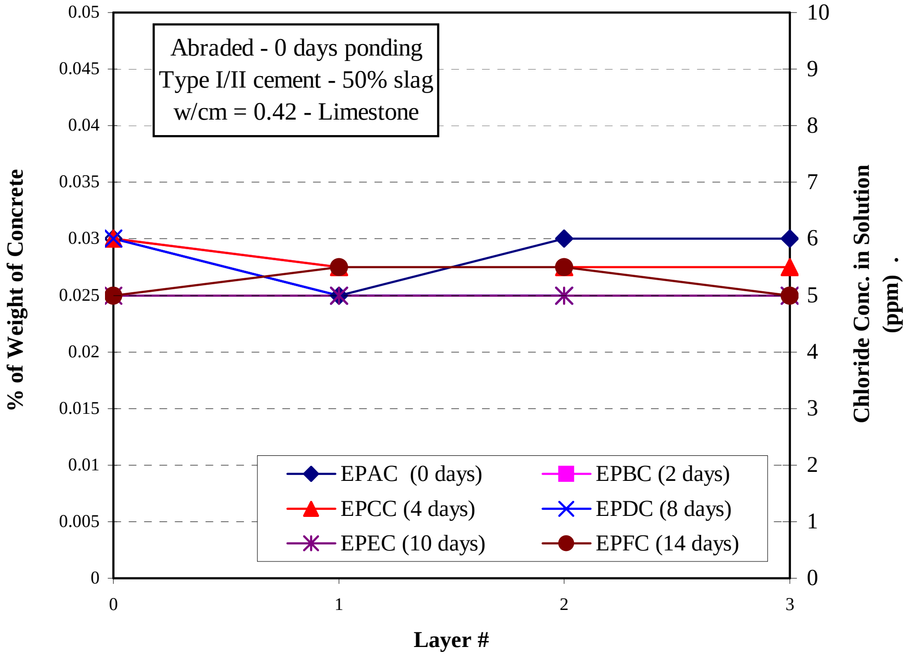


**Figure 4.21** Chloride Concentrations for El Paso Control Specimens

### 4.2.4 Atlanta Site

For the Atlanta site, concrete was mixed at water cementitious ratio of 0.46, and 21% of the required cement content was replaced with fly ash class F. The cement: fly ash: fine aggregate: coarse aggregate ratio was 1.00: 0.34: 2.92: 4.77 (volumetric). The coarse aggregate used was siliceous gravel. The chloride permeability of concrete from this site was tested after 180 days of ponding. The results of the Atlanta site are shown in Table 4.5. Within each column, as the curing duration increased, the chloride concentration decreased, thus the concrete became less permeable. Specifically, the chloride concentration decreased from 100 ppm for AA (0 day curing) to 75.5 ppm for AF (14 days curing).

**Table 4.5** Concentrations for Atlanta Ponding Specimens in ppm

| **Abraded – 180 days ponding – Type I/II Cement – 21% FA (class F) - Sil** | | | | |
|---|---|---|---|---|
| | 0-1/4” | ¼”-3/4” | ¾”-1 ¼” | 1 ¼”-1 ¾” |
| **AA (0 days)** | 100 | 40.5 | 8 | 3 |
| **AB (2 days)** | 92 | 31 | 4 | 2.5 |
| **AC (4 days)** | 79 | 23 | 4 | 2.5 |
| **AD (8 days)** | 78 | 22.5 | 3.5 | 3 |
| **AE (10 days)** | 78 | 21 | 3.5 | 2.5 |
| **AF (14 days)** | 75.5 | 21 | 3.5 | 2.5 |

Figures 4.22 and 4.23 show the relation between the chloride concentration and the curing duration. As shown, most of the reduction occurs during the first 4 days of curing, with no appreciable improvement in concrete permeability from longer curing durations.

The chloride concentration for the top quarter was extremely high (100 ppm or 20 lb/yd$^3$). This could be because of two reasons. First, in the case of fly ash, the products of reaction closely resemble C-S-H produced by hydration of Portland cement. However, this reaction does not start until some time after mixing, so the structure of the concrete is not well established and porous since the fly ash has not yet reacted (Neville 1996). With no curing, permeability is expected to be extremely high. The second reason is related to the use of siliceous gravel in the Atlanta mixture. It is well known that the use of gravel in a concrete mixture produces a relatively weak interface zone (Asbridge et al. 2001). This is due to the roughness and to the surface texture of the siliceous gravel.

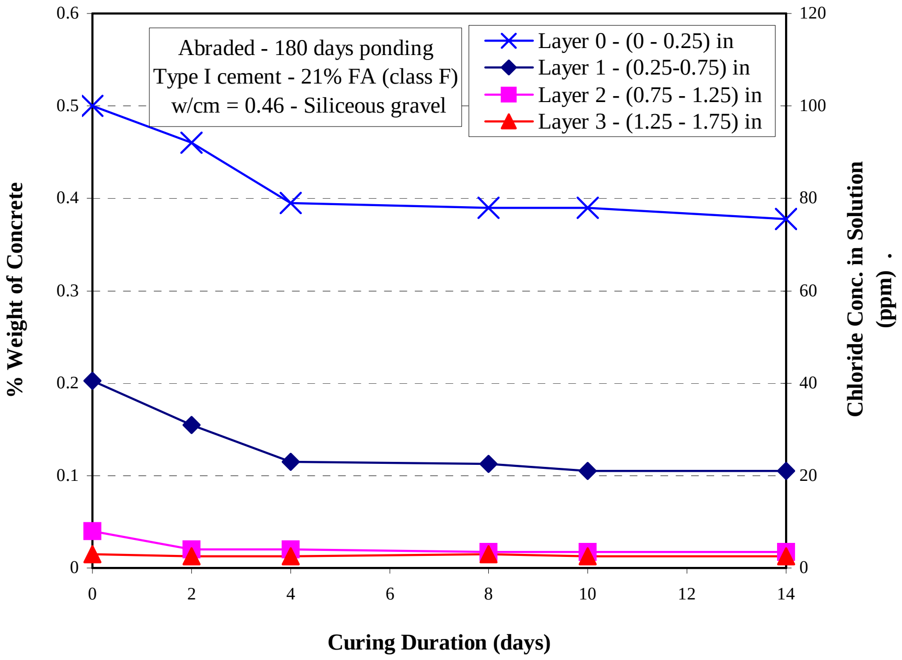


**Figure 4.22** Chloride Concentrations versus Curing Duration for Atlanta

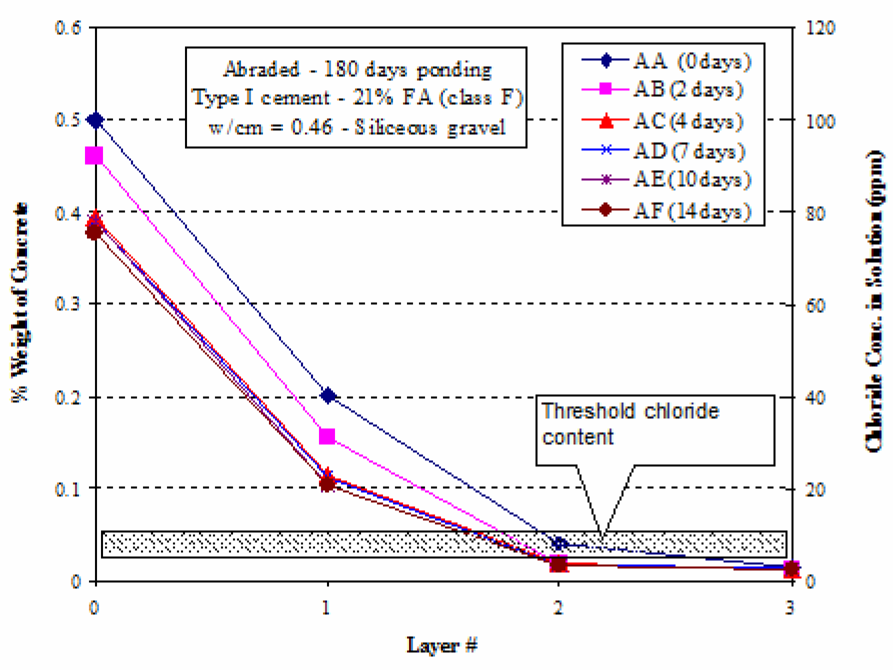


**Figure 4.23** Chloride Concentrations versus Layer Number for Atlanta

The results of the control samples for the Atlanta site are presented in Figure 4.24. As expected, chloride content was negligible in all layers for all curing durations except for the 0 day curing specimen.

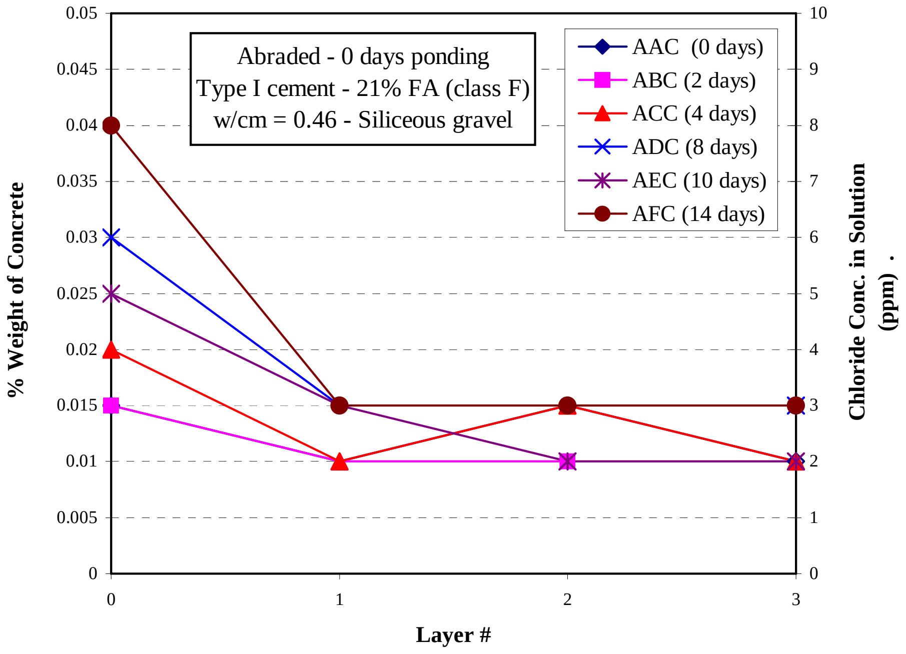


**Figure 4.24** Chloride Concentrations for Atlanta Control Specimens

4.2.5 San Antonio Site

For the San Antonio site, concrete was mixed at a water cement ratio of 0.43, and 20% of the required cement content was replaced with fly ash class C. The cement: fly ash: fine aggregate: coarse aggregate ratio was 1.00: 0.29: 2.68: 4.55 (volumetric). The coarse aggregate used was limestone. The chloride permeability of concrete from this site was tested after 180 days of ponding. Table 4.6 shows the results of the San Antonio specimens.

Unlike previous sites, the curing duration for this mixture showed no effect on the permeability of concrete for layer 0. That is, the chloride concentration for SAA was 64 ppm for 0 days curing, while the concentration for SAF was 62.5 ppm for 14 days curing. However, curing appears to have played a significant role in reducing the chloride ingress in layer 1 of concrete, as shown in Figure 4.25. While 0 day curing concrete showed high permeability, the 2, 4, 8, 10, and 14 cure days concrete showed much lower permeability.

**Table 4.6** Concentrations for San Antonio Ponding Specimens in ppm

| **Abraded – 180 days ponding – Type I/II Cement – 20% FA (class C) - LM** | | | | |
|---|---|---|---|---|
| | 0-1/4” | ¼”-3/4” | ¾”-1 ¼” | 1 ¼”-1 ¾” |
| **SAA (0 days)** | 64 | 36 | 6 | 3 |
| **SAB (2 days)** | 61 | 25.5 | 4.5 | 3 |
| **SAC (4 days)** | 64 | 25 | 4 | 3 |
| **SAD (8 days)** | 63 | 25.25 | 4 | 3 |
| **SAE (10 days)** | 63 | 25 | 4 | 3 |
| **SAF (14 days)** | 62.5 | 23 | 5 | 3 |

From the plot of Figure 4.25, increased curing duration showed no appreciable effect on permeability for all curing durations for layers 2 and 3. This was likely due to the fact that the interior of the concrete was hardly subjected to moisture movement, and thus no positive effect of curing on the interior of concrete occurred. In contrast, curing was extremely critical for the upper layers, where concrete was subjected to environmental conditions such as evaporation and temperature. In this case, curing provided the satisfactory moisture required to allow the continuation of hydration.

Figure 4.26 shows a relation between the chloride content and the layer number. As expected, most of the chloride was located in the upper two layers, decreasing as the depth level increased. This phenomenon could be due to the pore walls, which provided resistance to the flow of liquid. In general, the degree of this resistance depends on pore dimensions, with smaller pores presenting greater resistance than larger pores.

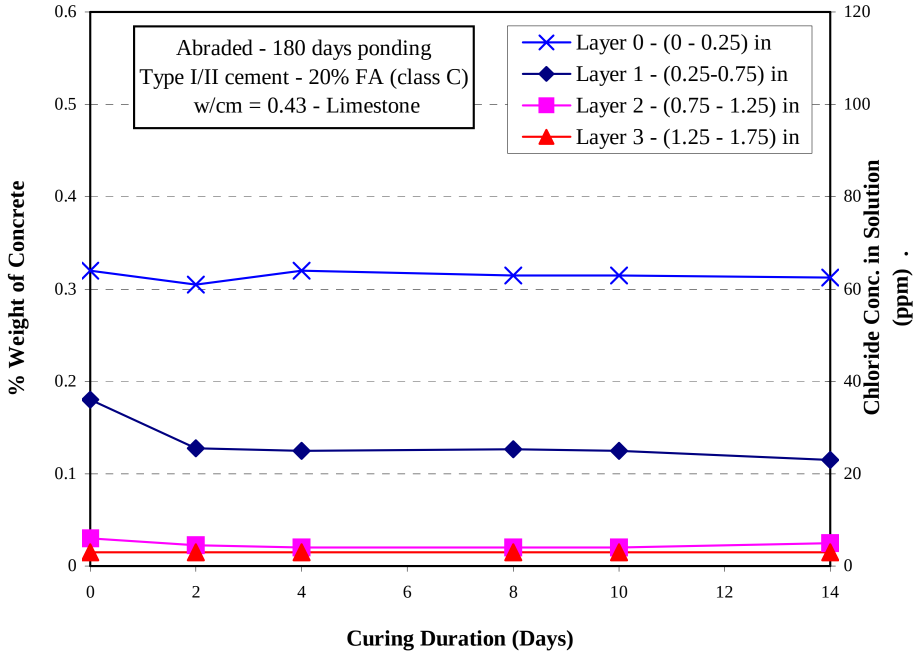


**Figure 4.25** Chloride Concentrations versus Curing Duration for San Antonio

For the San Antonio site, the chloride content for layer 2 fell below the threshold chloride content as shown in the shaded area in Figure 3.14, so the steel reinforcement can be put at a depth of 1.25” below the top surface of the concrete. Typically, in bridge deck, steel reinforcing bars are specified with a clear cover of 2” or more.

The complete test results for the control samples of San Antonio are shown in Figure 4.27. Similar to previous sites, the amount of chloride was extremely negligible for all layers and curing durations except for the 0 day curing specimen, where the concentration reached 7 ppm in solution. This is an extremely high number for a control specimen. This high number may be due to the fact that at 0 days curing, the top surface prematurely dried. To maintain equilibrium in the sample, water carrying chloride ions likely moved toward the top surface and evaporated, leaving the chloride ions at the top surface. For longer curing durations, the concrete top surface became partially saturated, and no movement of water toward the top surface occurred. Thus, lower chloride concentrations were found.

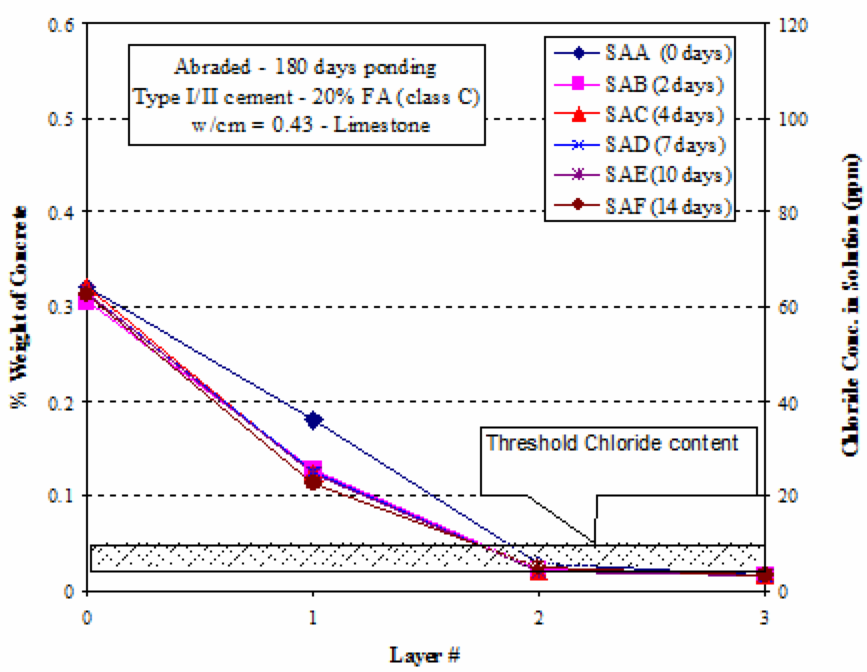


**Figure 4.26** Chloride Concentrations versus Layer Number for San Antonio

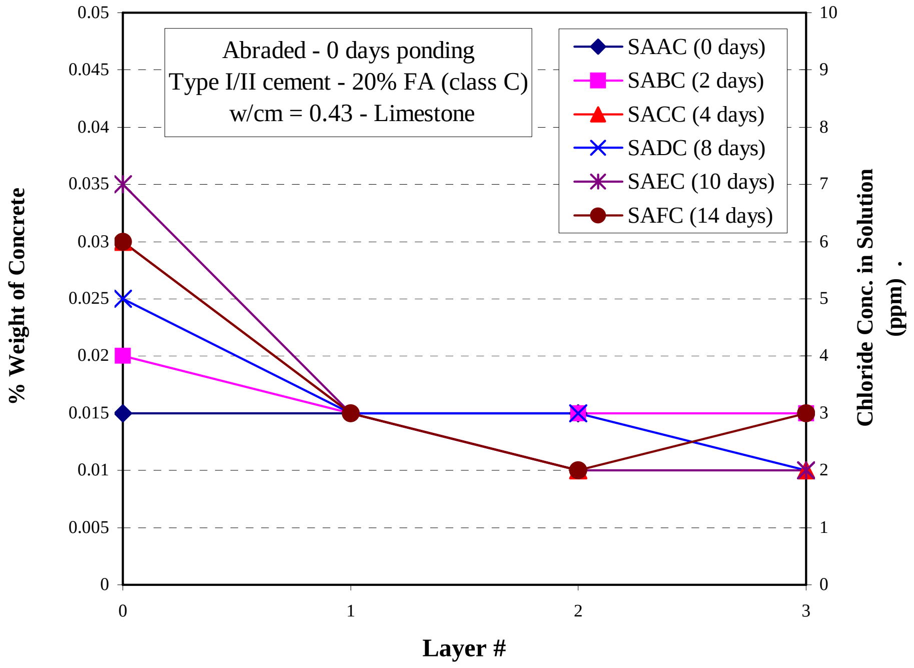


**Figure 4.27** Chloride Concentrations for San Antonio Control Specimens

### 4.2.6 Pharr Site

For the Pharr site, concrete was mixed at a water cementitious ratio of 0.41, and no mineral admixture was used. The cement: fly ash: fine aggregate: coarse aggregate ratio was 1.00: 0.00: 1.95: 3.97 (volumetric). The coarse aggregate used was siliceous gravel. The chloride permeability of concrete from this site was tested after 180 days of ponding. Results from the Pharr site tests are shown in Table 4.7.

From the plot of chloride concentration versus curing duration, the initial concentration at 0 day curing was higher for the Pharr mixture than for any other concrete mixture tested; that is, the chloride concentration decreased from 41 ppm  for HAA (layer 1, 0 day curing) to 28 ppm for HAF (layer 1, 14 days curing). These results indicate that curing helps the hydration of cement particles. As the curing duration increased, more calcium silicate hydrate likely was formed. Thus voids in the concrete structure tended to become discontinuous and smaller, and the permeability of the concrete was reduced.

**Table 4.7** Concentrations for Pharr Ponding Specimens in ppm

| **Abraded – 180 days ponding – Type I Cement – No mineral admixture - Sil** | | | | |
|---|---|---|---|---|
| | 0-1/4” | ¼”-3/4” | ¾”-1 ¼” | 1 ¼”-1 ¾” |
| **HAA (0 days)** | 64 | 36 | 6 | 3 |
| **HAB (2 days)** | 61 | 25.5 | 4.5 | 3 |
| **HAC (4 days)** | 64 | 25 | 4 | 3 |
| **HAD (8 days)** | 63 | 25.25 | 4 | 3 |
| **HAE (10 days)** | 63 | 25 | 4 | 3 |
| **HAF (14 days)** | 62.5 | 23 | 5 | 3 |

Figures 4.28 and 4.29 indicate the same set of relations between the chloride concentration and the curing duration for the four depths under study as presented previously for the other test mixtures. Chlorides were concentrated in the upper two layers (up to 0.75”). In contrast, chloride content was negligible in the lower two layers and almost negligible in the bottom layer. Pore wall resistance likely played a principal role in the decline of chloride concentration through the depth of the specimen (Neville 1996).

Interestingly, the curing duration does not appear to play a significant role with respect to the permeability of the Pharr mixture concrete after 8 days. As shown in Figure 4.28, while chloride concentration decreased 93% [(90-75)/ (90-74)] when curing increased from 0 to 8 days, it only decreased by 7% [(75-74)/ (90-74)] after that, which led us to conclude that 8 days of curing was enough for the Pharr site.

The hatched area in Figure 4.29 represents the threshold content to initiate the corrosion in concrete structures exposed to chloride in service. As seen from the plot, the top two layers are not the appropriate location since all numbers for all curing durations are much higher than the threshold content. However, based on the plot, steel placed below 1.25" from the top surface would be a location relatively immune to chloride attack.

Results from the control specimens for the Pharr site are presented in Figure 4.30. Similar to previous site results, the amount of chloride appears to be distributed randomly across the specimen. Also, for the control specimens, curing does not appear to have a significant effect on the concrete permeability as the chloride content ranges from 1 to 3 ppm for all curing durations.

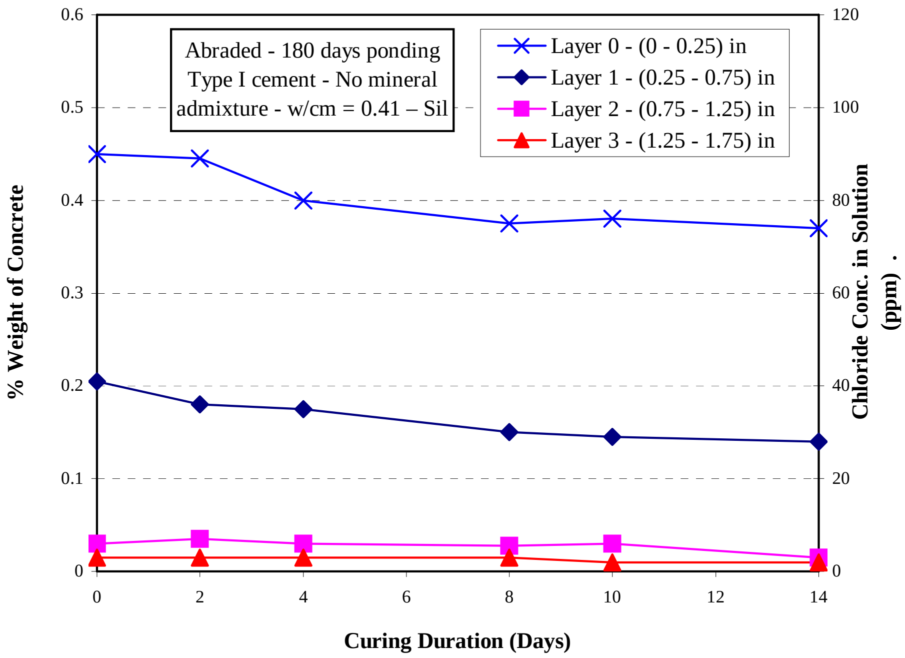


**Figure 4.28** Chloride Concentrations versus Curing Duration for Pharr

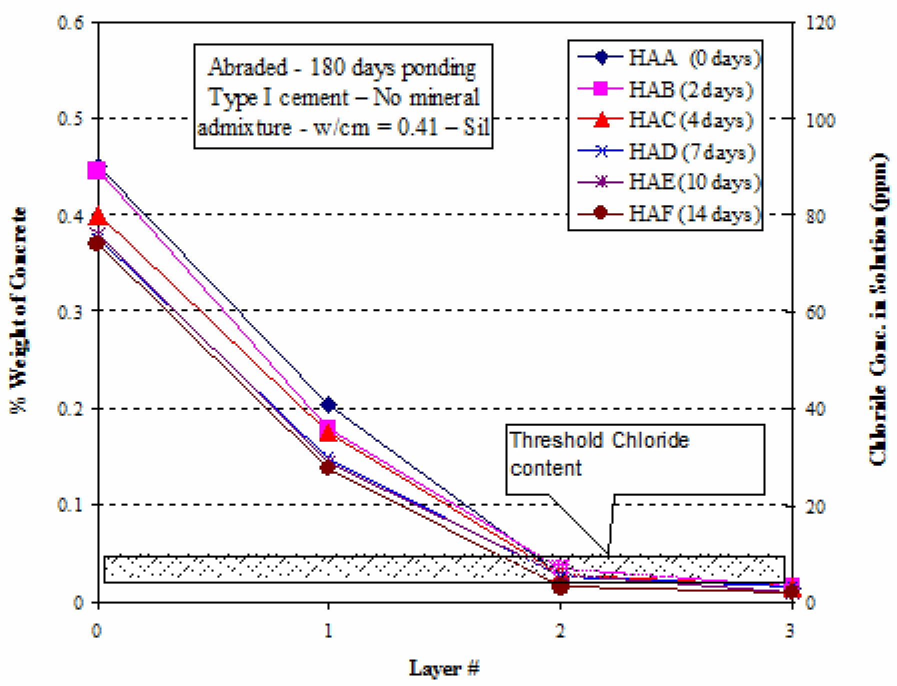


**Figure 4.29** Chloride Concentrations versus Layer Number for Pharr

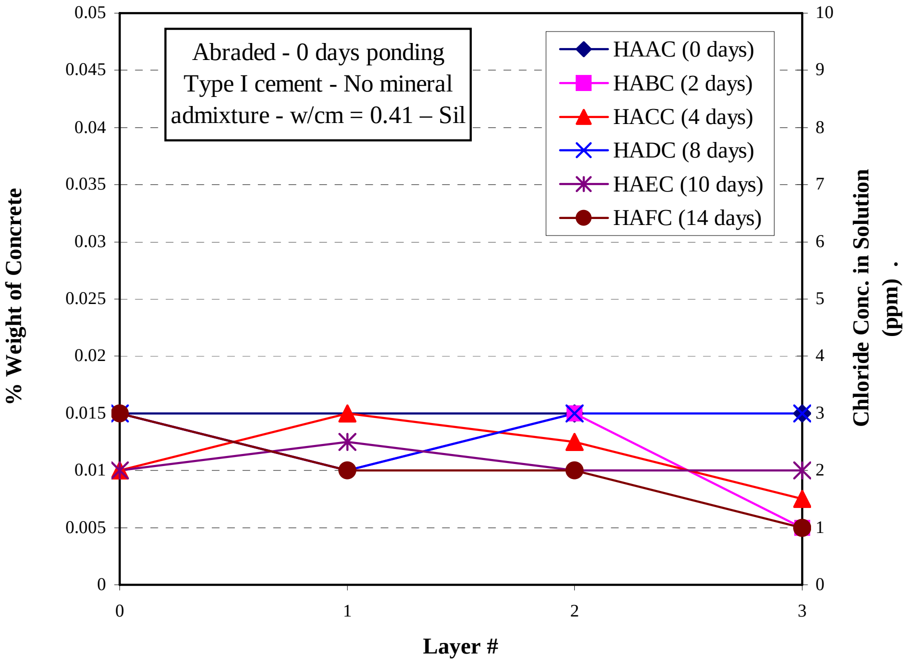


**Figure 4.30** Chloride Concentrations for Pharr Control Specimens

### 4.2.7 Houston Site

For the Houston site, concrete was mixed at water cementitious ratio of 0.46, and 27% of the required cement content was replaced with fly ash class C. The cement: fly ash: fine aggregate: coarse aggregate ratio was 1.00: 0.43: 3.04: 5.33 (volumetric). The coarse aggregate used was limestone. The chloride permeability of concrete from this site was tested after 180 days of ponding. The results of the Houston site are shown in Table 4.8 and Figure 4.31.

For this mixture, it is evident that the length of curing duration plays a major role in reducing the concrete permeability. As can be seen from Figure 4.31, the permeability of 0 day cure concrete was significantly higher than the permeability of any other concrete subjected to another curing duration. In addition, after 4 days of curing, no significant improvement in concrete permeability occurred. That is, while the chloride concentration decreased from 60 ppm for HA (level 0, 0 day curing) to 45 ppm for HC (level 0, 4 days curing), it decreased by just 3.5 ppm after that. Similar to other test mixtures, no effect of curing was seen in layers 2 or 3 on the permeability of the concrete.

**Table 4.8** Concentrations for Houston Ponding Specimens in ppm

| Abraded - 180 days ponding - Type I/II cement - 27% Fly ash (class C) - LM | | | | |
|---|---|---|---|---|
| | 0 - ¼” | ¼” - ¾” | ¾” - 1¼” | 1¼”- 1¾” |
| **HA (0 days)** | 60 | 31 | 5.5 | 2.5 |
| **HB (2 days)** | 54 | 27 | 4.5 | 2.25 |
| **HC (4 days)** | 45 | 23 | 4 | 2.5 |
| **HD (8 days)** | 44 | 22 | 4 | 2 |
| **HE (10 days)** | 43 | 21.5 | 4.25 | 1.5 |
| **HF (14 days)** | 41.5 | 21 | 3 | 1.5 |

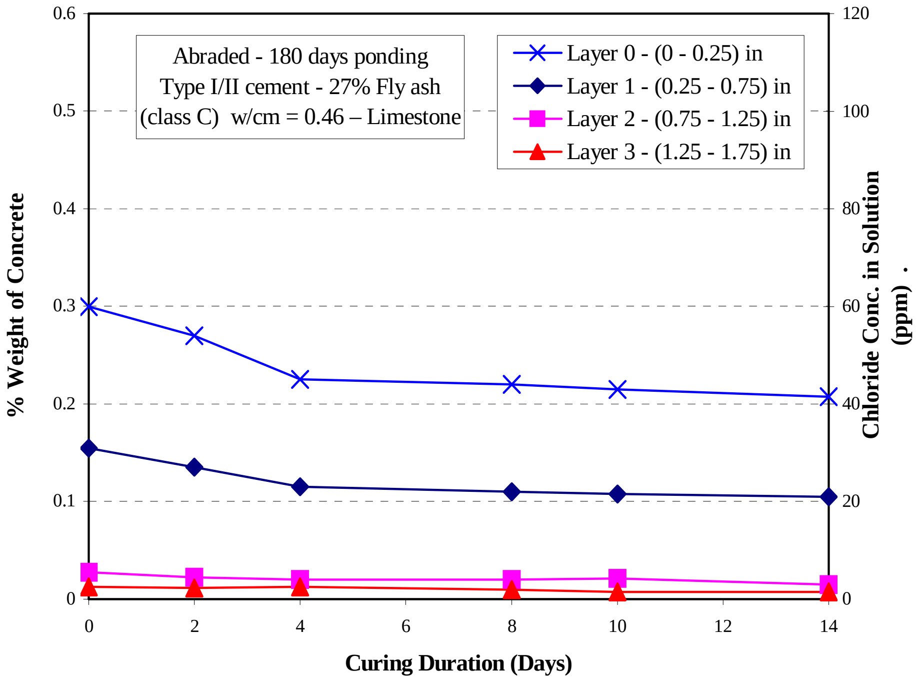


**Figure 4.31** Chloride Concentrations versus Curing Duration for Houston

The relation between the chloride content and the layer number with respect to the threshold level is shown in Figure 4.32. The huge decrease in chloride concentration shown is likely due to the resistance of the pore walls as they prevent further chloride to penetrate through the concrete. In general, small pores provide higher resistance than larger ones. The hatched area represents the threshold chloride content necessary to initiate the corrosion. All chloride concentrations for the top two layers were much higher than the values of the threshold, leading one to assume that the placement level of reinforcement should be greater than 1.25” from the top surface of the concrete.

Control specimen results for the Houston site are shown in Figure 4.33. Similar to all previous sites, negligible amounts of chloride are inherently found in the concrete. These small amounts are likely due to a certain amount of chloride found in the mixing water and/or aggregates.

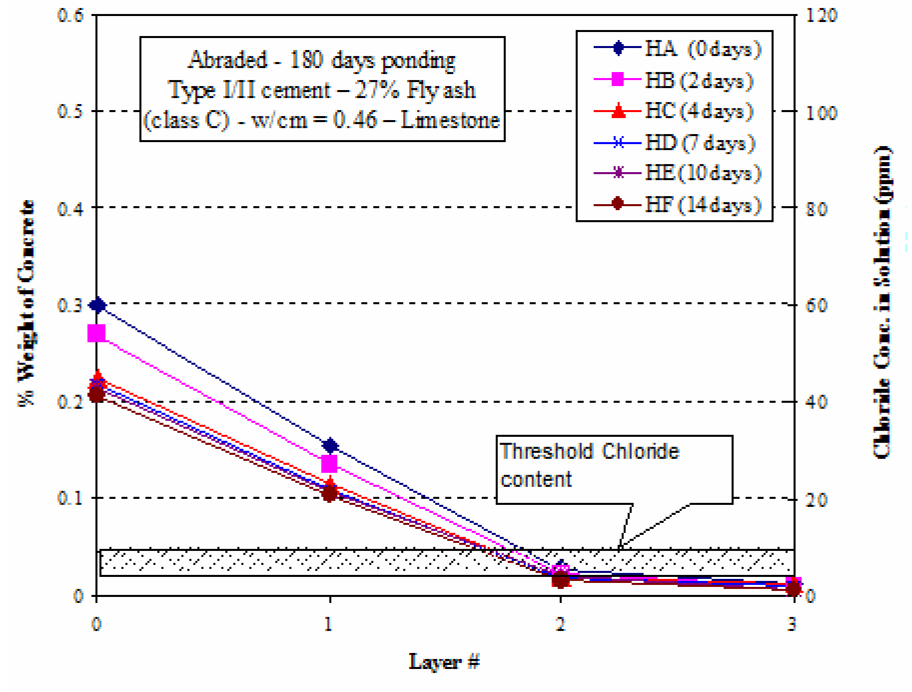


**Figure 4.32** Chloride Concentrations versus Layer Number for Houston

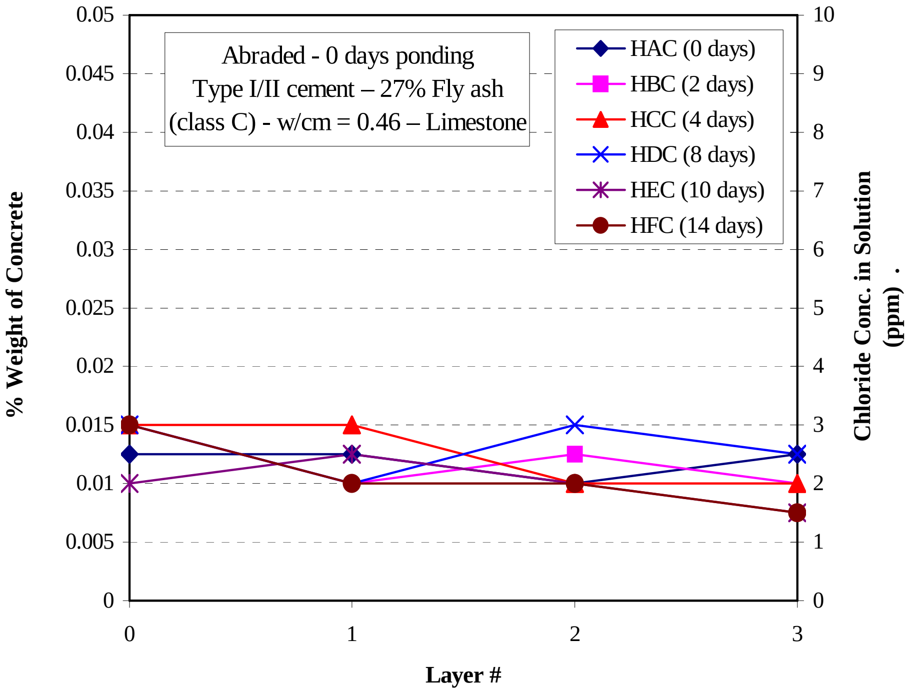


**Figure 4.33** Chloride Concentrations for Houston Control Specimens

## 4.3 Discussion

Analysis of ponding test results show that the mix design that did not have the pozzolanic material (i.e. Atlanta) had by far the highest chloride concentration at the top ¼ inch of the slab. However, chloride concentration at ¼ inch to ¾ inch depth did not show significant differences.

Using 50% granulated blast furnace slag as a cement replacement material (in the El Paso site mixture, see Figure 4.19), it was found that concrete permeability is quite high compared to the other sites subjected to 180 days of ponding. Whereas with the use of fly ash (Fort Worth in Figure 4.13 versus New Lubbock in Figure 4.16; San Antonio in Figure 4.25 versus Houston in Figure 4.31), lower values of permeability were obtained. It was also observed that as the initial wet-mat curing period increased, fly ash concrete became considerably more impermeable than fly ash concrete cured for shorter durations.

In general, the duration of wet-mat curing has been found to be an important criterion in reducing the permeability of concrete. From the results of the seven sites, it has been found that 2 days of curing for El Paso and San Antonio, 4 days of curing for New Lubbock, Atlanta, and Houston, and 8 days of curing for Pharr and Fort Worth provided the effective reduction in permeability of concrete.

Sites that showed the highest permeability were Pharr and Atlanta. This may be due to the fact that siliceous gravel and Type I cement were used for these two mixtures. Siliceous gravel, when used in concrete mixes, leads to the formation of a weak interface zone.

### 4.3.1 90-Day Ponding Comparison: Fort Worth versus New Lubbock

A comparison was made between the Fort Worth and New Lubbock mixtures and summary of this comparison shown in Figures 4.34 through 4.37. Both sites were ponded for 90 days and contain 22% and 31% fly ash class F, respectively (Table 4.2). As the figures show, as the percentage of fly ash increased from 22% to 31%, concrete permeability decreased by almost 40% for layers 0 and 1 for up to 4 days of curing. This decrease was likely due to the fact that more fly ash reacted with the calcium hydroxide produced during cement hydration to form additional calcium silicate hydrate, which fills large capillary voids and disrupts their continuity

(Alhozaimy et al. 1996). The blocking of capillary voids reduces the permeability of concrete. Another reason for this reduction could be that fly ash had influenced the microcracking process at the aggregate paste interface by altering the volume and shrinkage characteristics of the paste, as well as by changing the bleeding properties of cement paste and the microstructure at the interface zone (Neville 1996).

This comparison also leads to the conclusion that curing duration plays an important role in determining the permeability of concrete. After 8 days of curing, permeability of New Lubbock is slightly lower than that of Fort Worth (22.5%, layer 0, Figure 4.34).

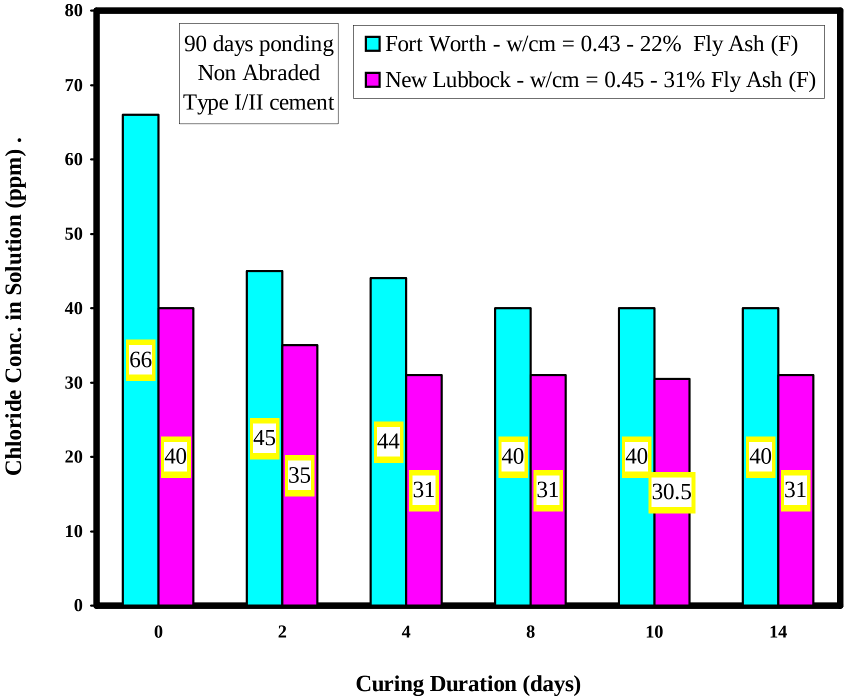


**Figure 4.34** Comparison of Fort Worth versus New Lubbock (layer 0)

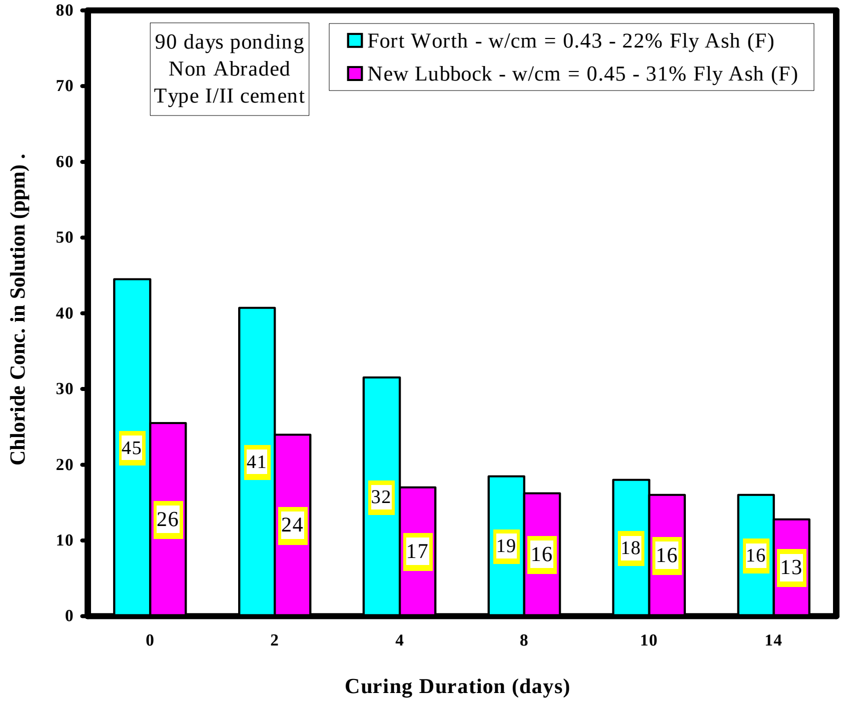


**Figure 4.35** Comparison of Fort Worth versus New Lubbock (layer 1)

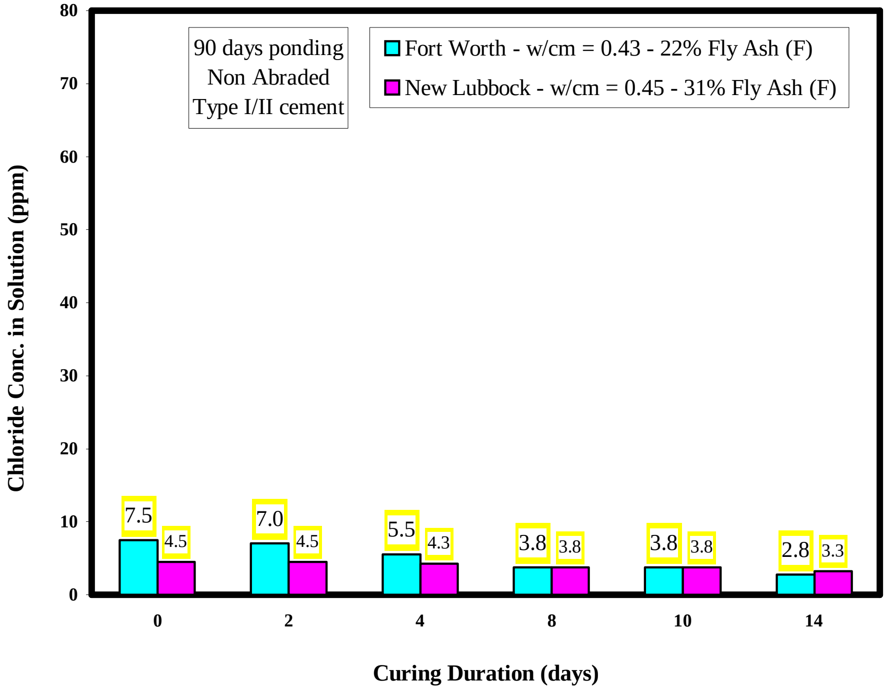


**Figure 4.36** Comparison of Fort Worth versus New Lubbock (layer 2)

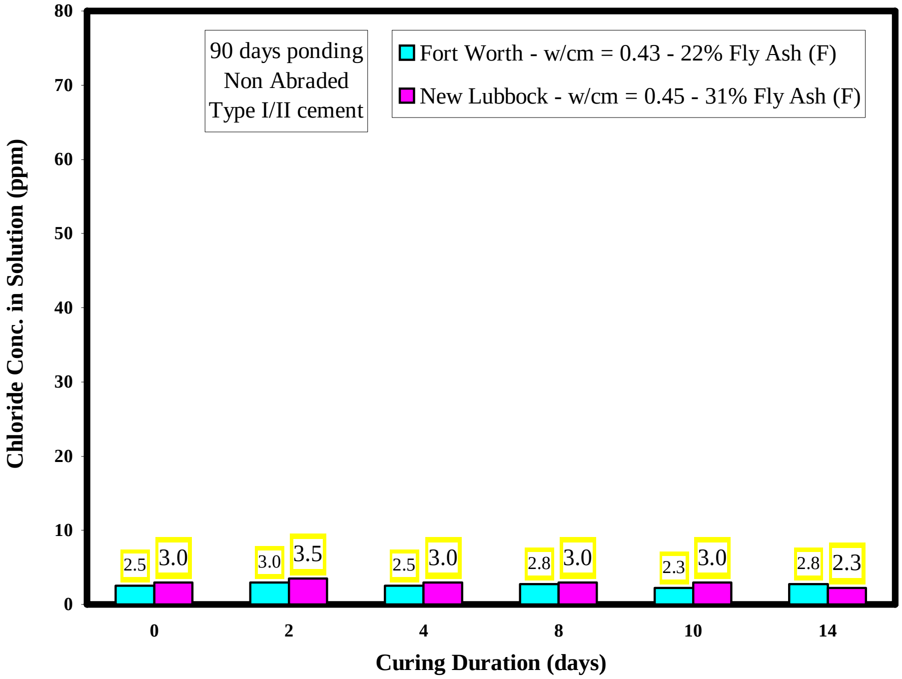


**Figure 4.37** Comparison of Fort Worth versus New Lubbock (layer 3)

4.3.2 180-Day Ponding Comparison: San Antonio versus Houston

An additional direct comparison between the Houston and San Antonio sites is shown in Figures 4.38 through 4.41. Both specimens had been ponded for 180 days and contained 27% and 20% class C fly ash respectively. Similar to New Lubbock on the previous two site mixtures, the permeability test results for the Houston mixture were lower than those of the San Antonio site for layer 0 and 1. Again, this likely was due to the fact that the higher percentage of fly ash in the Houston mixture resulted in a finer pore structure in both the cement paste matrix and along the paste aggregate interface in the concrete.

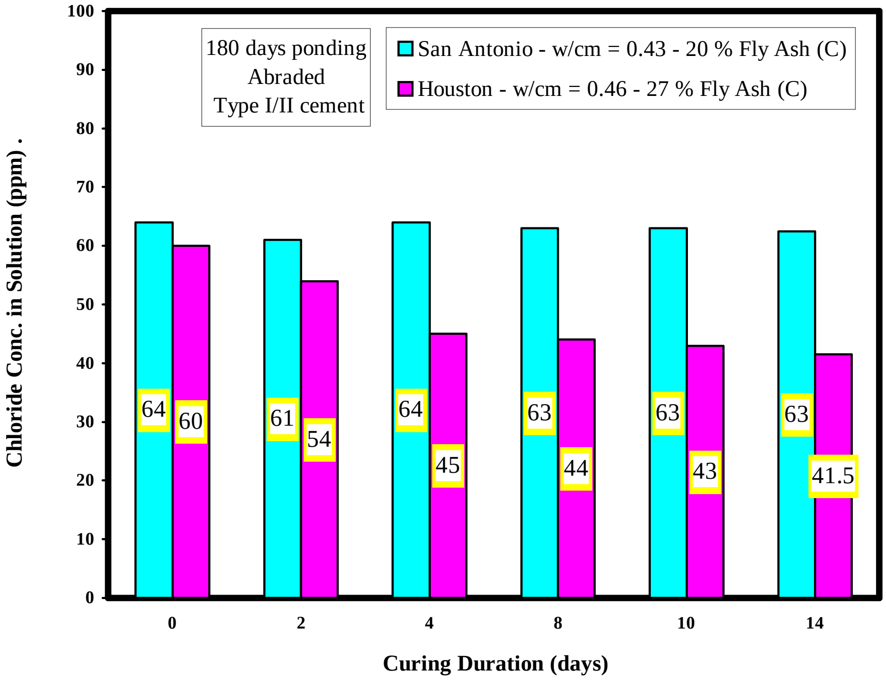


**Figure 4.38** Comparison of San Antonio versus Houston (layer 0)

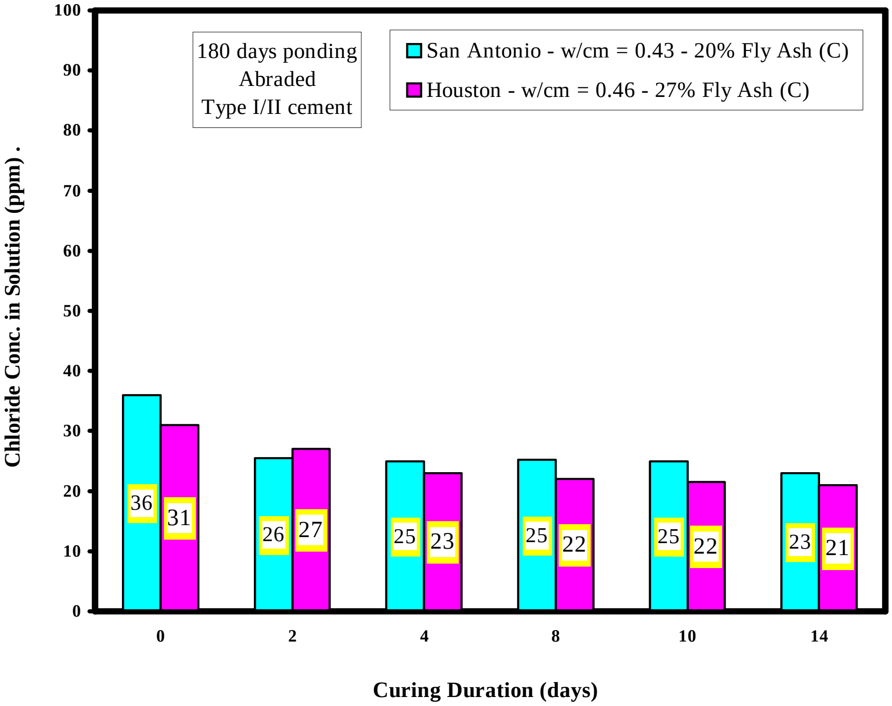


**Figure 4.39** Comparison of San Antonio versus Houston (layer 1)

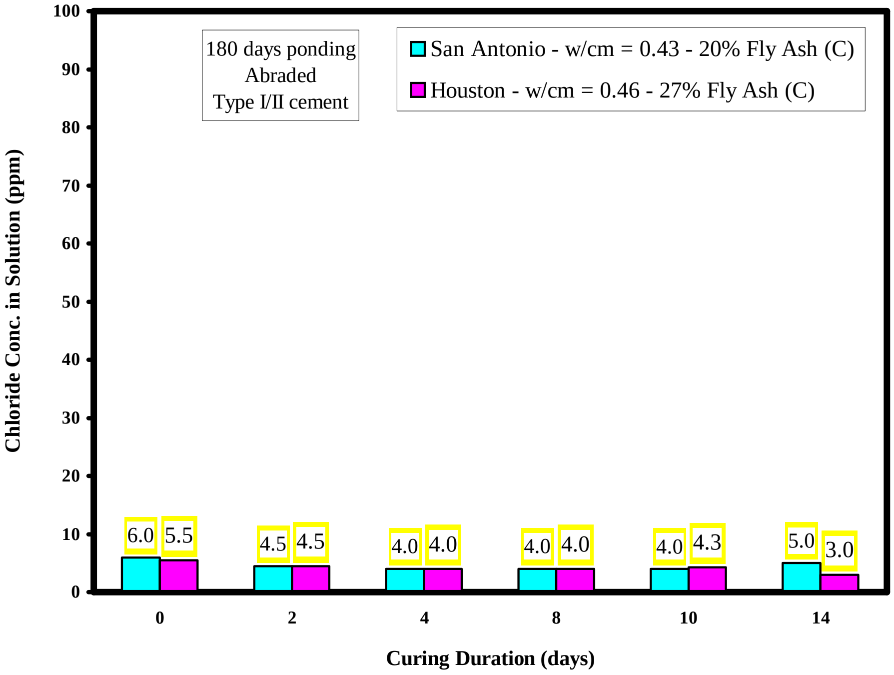


**Figure 4.40** Comparison of San Antonio versus Houston (layer 2)

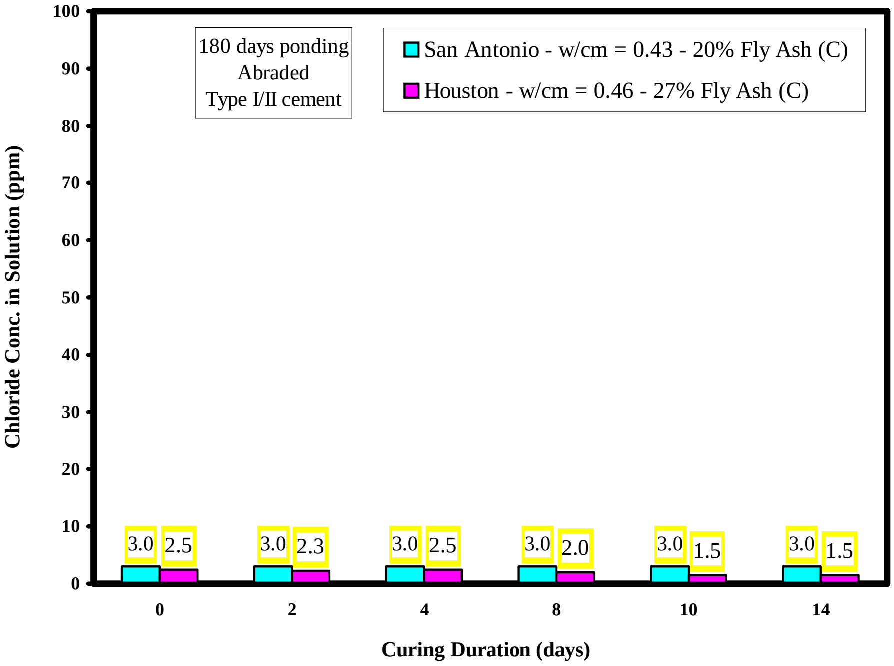


**Figure 4.41** Comparison of San Antonio versus Houston (layer 3)

### 4.3.3 Discussion of Direct Comparisons

The duration of ponding and the nature of the top surface of the concrete (whether it is abraded or not) appear to play a major role in determining the permeability of the concrete. It can be seen that the Houston and San Antonio mixtures have higher permeabilities than the Fort Worth and New Lubbock mixtures. This could be due to two reasons. The first is that the Fort Worth and New Lubbock mixtures were ponded for only 90 days each, whereas the Houston and San Antonio mixtures were ponded for 180 days; more time was allowed for the chloride ions to diffuse through the concrete and overcome the resistance of the pore walls. Second, the top surfaces of the Fort Worth and New Lubbock site specimens were not abraded, while the Houston and San Antonio site specimens were abraded. It is highly probable that the thin layer at the top surface of the concrete (when it was not abraded) provided protection for the concrete. In fact, the non-abraded surface appeared to provide a shielding effect for the top surface, avoiding direct contact between the chloride ions in the solution and the interface zone at the top surface of the concrete.

### 4.3.4 180-Day Ponding Comparison: Pharr versus Atlanta

Siliceous gravel, well known for its smooth surface texture, leads to the formation of a weak bond between the aggregate and the cement paste. This weak bond leads to the formation of a weak interface zone, which increases the permeability of concrete by providing channels for the chloride ions to pass. This fact may be the primary reason why sites such as Pharr and Atlanta with siliceous gravel show higher permeability results than sites with fly ash such as Fort Worth, New Lubbock, San Antonio, Houston, and El Paso.

A comparison made between the Pharr and Atlanta site mixtures is shown in Figures 4.42 and 4.43. Both sites have siliceous gravel and were ponded for 180 days. The main difference between them is that Pharr has no mineral admixtures added, while the Atlanta site mixture had 21% of the cement replaced with class F fly ash.

An interesting point was observed in Figure 4.43. Initially, the permeability values were the same shown for Atlanta for 0 days of curing. However, as the curing duration increased, the permeability of the Pharr mixture became greater than that of the Atlanta mixture.

This phenomenon could be due to the following reason: in the presence of fly ash, the chemical reaction became very slow. A consequence of this delay in the reaction of fly ash is the beneficial pattern of heat evolution. However, this delayed heat generation can lead to a higher permeability in the first couple of days in comparison with a mix that does not have fly ash, since most hydration products had not been formed in this short amount of time. This explains the higher permeability number for Atlanta in the first few days of ponding. At a later stage, with the presence of fly ash, more hydration products diffused away and precipitated within the capillary pore system; this resulted in a reduction in the capillary porosity and consequently in a finer pore structure and lower permeability number (Asbridge et al. 2001). This could explain the lower permeability values for Atlanta after 2 days of curing (Figure 4.43).

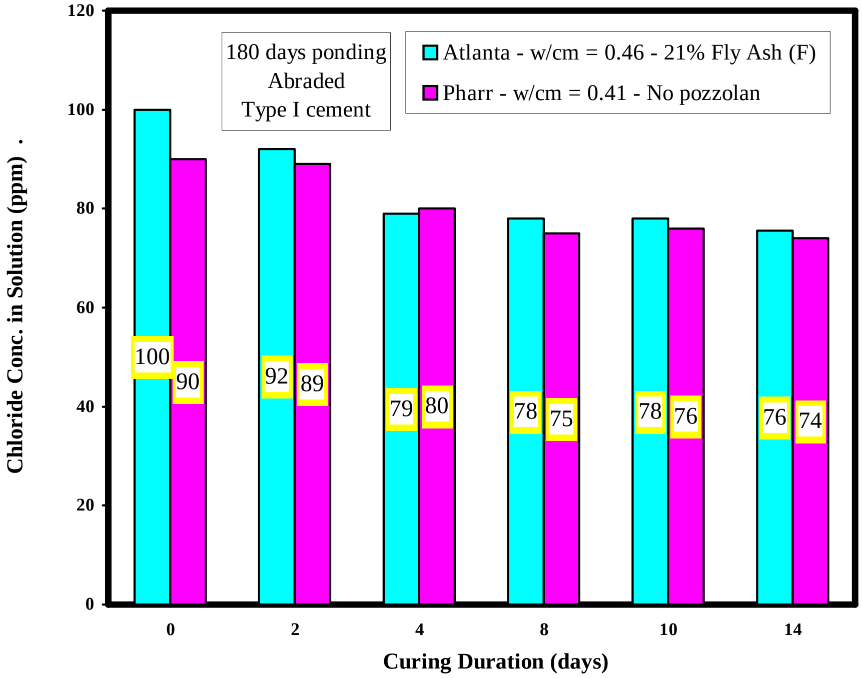


**Figure 4.42** Comparison of Pharr versus Atlanta (layer 0)

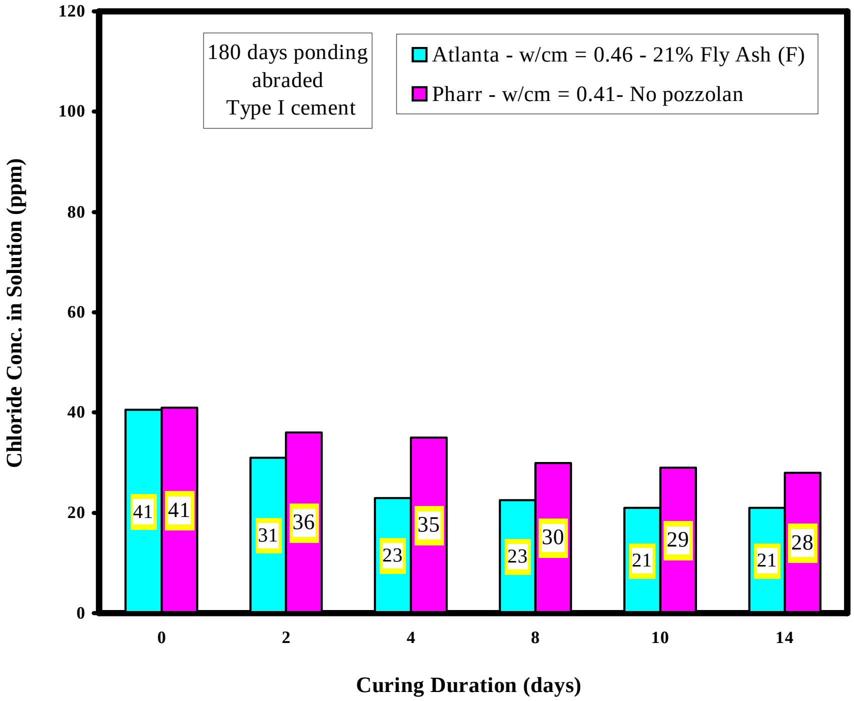


**Figure 4.43** Comparison of Pharr versus Atlanta (layer 1)

# CHAPTER V

# COMPARISON OF RCPT AND PONDING TEST RESULTS

## 5.1 Discussion of the Comparison between AASHTO T277 and AASHTO T259/260

Whiting (1981) introduced the total integral chloride *I*. This value was calculated using the acid soluble chloride content at 0.2” (5.1 mm) intervals from the surface of the ponded specimen to a depth of 1.6” (41 mm). This method allowed chloride penetration data from the 90 day ponding test to be presented as a single value. However, in this research project, only the water soluble chloride was determined because the focus was to ascertain the free chloride ions responsible for the corrosion. Thus, the comparison was made between the ponding test (water soluble chloride) and the Rapid Chloride Permeability Tests (RCPT).

The results of the RCPT for the seven sites are presented in Table 5.1. Using the permeability definitions presented and discussed in the FHWA report, the mixtures were classified based upon their water soluble integral chloride, illustrated in Table 5.2. Figures 5.1, 5.2, and 5.3 show the classification of the seven sites for 4, 8, and 10 days of curing. To make a comparison with Whiting’s original research, all mixtures with 14 days of curing were classified based on their water soluble integral chloride, and their coulomb values as shown in Table 5.3. Three sites (Pharr, Atlanta, and Fort Worth) exhibited identical water soluble integral chloride and coulomb classifications, while the other four sites showed some discrepancies. For example, the El Paso and San Antonio mixtures exhibited moderate water soluble integral chloride while exhibiting very low coulomb values. The New Lubbock mixture exhibited very low water soluble integral chloride values, while exhibiting low coulomb values. The Houston mixture exhibited low integral chloride values, while exhibiting very low coulomb values. As a result, it is concluded that the two tests are not always consistent in predicting the permeability levels defined in the original test development report of AASHTO T277. However, it is important to note that in the case of this study, almost all of the AASHTO T277 results ended in a lower classification than the ponded test results in sites ponded for 90 and 180 days. Most discrepancies arose from the fact that the total integral chloride resulted in a higher classification.

**Table 5.1** AASHTO T277 Test Results (Afroze 2002)

| Curing Duration | El Paso | Fort Worth | San Antonio | New Lubbock | Houston | Atlanta | Pharr |
|---|---|---|---|---|---|---|---|
| 0 | 3170.4 | 2411.3 | 1266.2 | 933.4 | 644.2 | 3572.5 | 3189.6 |
| 2 | 769.9 | 1624.5 | 1232.2 | 1024.8 | 716.6 | 3257.2 | 2364.6 |
| 4 | 617.0 | 1734.2 | 831.4 | 1222.0 | 784.0 | 2612.4 | 3088.8 |
| 8 | 567.4 | 1470.2 | 773.7 | 1176.3 | 820.6 | 2713.5 | 2502.5 |
| 10 | 595.9 | 1303.6 | 861.9 | 1057.0 | 750.9 | 3091.4 | 2999.1 |
| 14 | 400.5 | 1672.7 | 696.4 | 1126.5 | 969.9 | 2412.6 | 2607.7 |

**Table 5.2** Classification of Sites Based on Water Soluble Integral Chloride

| Curing Duration | El Paso | Fort Worth | San Antonio | New Lubbock | Houston | Atlanta | Pharr |
|---|---|---|---|---|---|---|---|
| 0 | 1.18 | 1.04 | 0.96 | 0.66 | 0.86 | 1.27 | 1.19 |
| 2 | 0.91 | 0.95 | 0.79 | 0.62 | 0.76 | 1.04 | 1.13 |
| 4 | 0.90 | 0.79 | 0.80 | 0.50 | 0.65 | 0.86 | 1.05 |
| 8 | 0.93 | 0.61 | 0.80 | 0.48 | 0.63 | 0.85 | 0.95 |
| 10 | 0.92 | 0.58 | 0.79 | 0.48 | 0.61 | 0.83 | 0.94 |
| 14 | 0.90 | 0.54 | 0.78 | 0.42 | 0.58 | 0.81 | 0.88 |

**Table 5.3** Classification of Sites Based on Water Soluble Integral Chloride and Coulomb Values

| | El Paso | Fort Worth | San Antonio | New Lubbock | Houston | Atlanta | Pharr |
|---|---|---|---|---|---|---|---|
| Water Soluble Integral Chloride | 0.9 | 0.54 | 0.77 | 0.42 | 0.57 | 0.81 | 0.87 |
| | Moderate | Low | Moderate | Very Low | Low | Moderate | Moderate |
| Coulombs value | 400.5 | 1672 | 696 | 1126 | 969 | 2412 | 2607 |
| | Very Low | Low | Very Low | Low | Very Low | Moderate | Moderate |

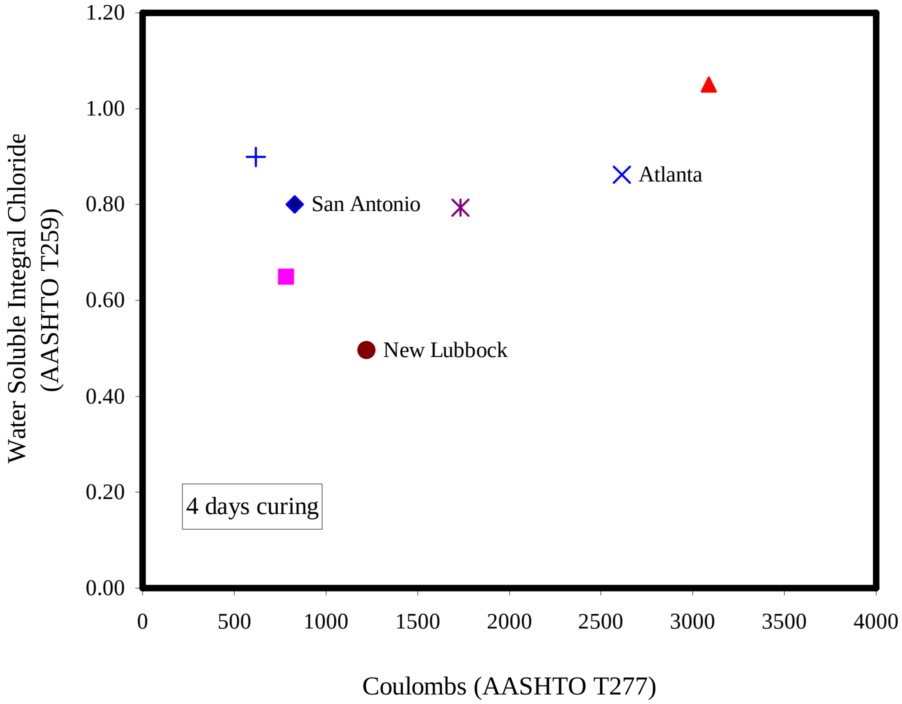


**Figure 5.1** Classifications of Concrete Mixtures for 4 Days of Curing

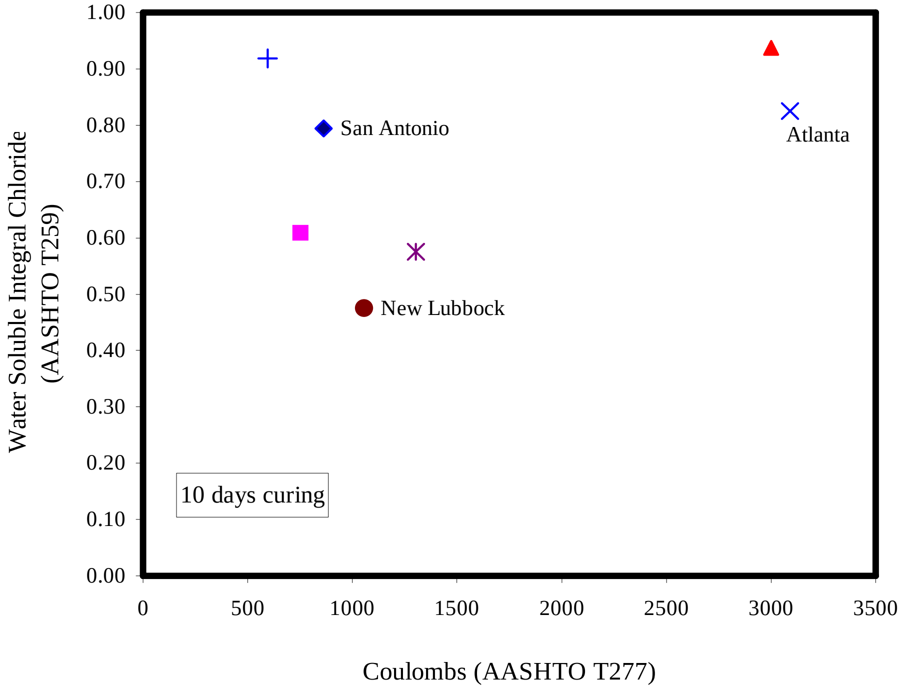


**Figure 5.2** Classifications of Concrete Mixtures for 8 Days of Curing

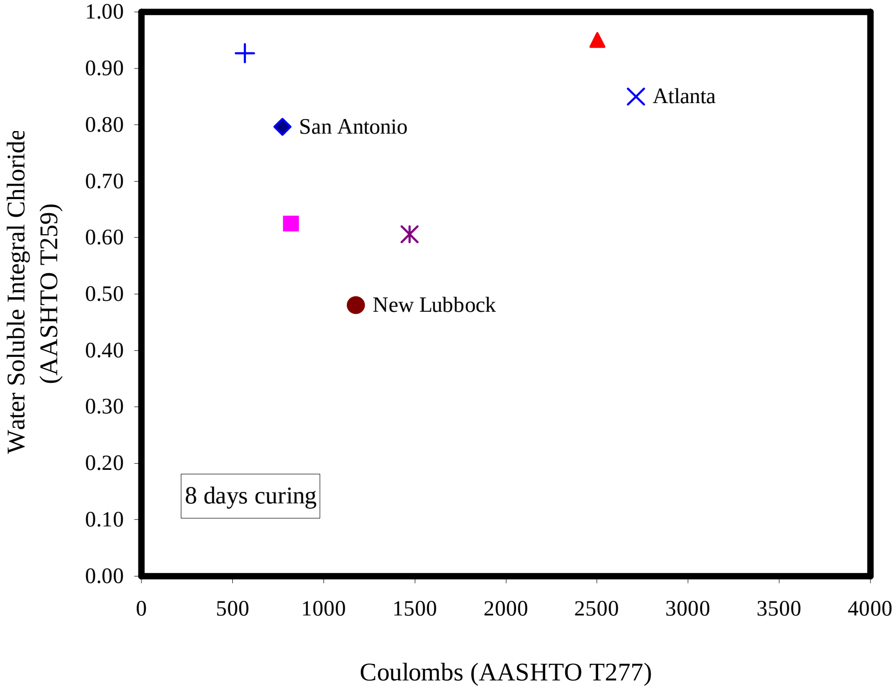


**Figure 5.3** Classification of Concrete Mixtures for 10 Days of Curing

## 5.2 Linear Regression Analysis

The correlation between AASHTO T277 and AASHTO T259 for this experimental program is investigated further by performing a linear regression analysis on three sets of data. Three cases are considered: 1) the water soluble integral chloride values – *I* values – were coupled with their corresponding coulomb values for the sites subjected to 180 days of ponding for 14 days of curing; 2) the 180 day chloride results in percent weight of concrete for the top ¼” were coupled with their corresponding coulomb values for 14 days of curing. It should be noted that El Paso results were not included in the correlation for both methods because of the inconsistent classification shown in Table 5.3 the water soluble integral chloride values – *I* values – were coupled with their corresponding coulomb values for all sites and for all curing durations. Results of these three regressions are shown in Figures 5.4, 5.5, and 5.6. As shown, the uppermost ¼” resulted in an $R^2$ correlation coefficient of 0.625 (Figure 5.5) and exhibited a better correlation to the T277 results than the water soluble integral chloride, which displayed an $R^2$ coefficient of 0.478 (Figure 5.4). In the original FHWA report, the authors used the total integral chloride and reported an $R^2$ correlation coefficient of 0.70 (Whiting 1981). However, the correlation is slightly lower in this case, possibly due to the fact that our mix includes mineral admixtures while all the mixes tested in Whiting's research do not contain mineral admixtures.

Figure 5.3 shows that the correlation of AASHTO T277 and AASHTO T260 for Atlanta, Fort Worth, and San Antonio are relatively good ($R^2$ = 0.73, 0.56, and 0.45 respectively). However, the correlations for New Lubbock and Houston mixtures are weak. This is due to the inconsistent results obtained for the RCPT in these two sites. That is, the 0 day coulombs for both sites are higher than the coulomb values for the 14 day curing concrete (Table 5.1), which could lead to the unrealistic conclusion that non-cured concrete has better permeability than well-cured concrete. The inconsistent results for the Houston mixture are due to the fact that two different concrete mixes with two different compressive strengths were used during the mixing. The New Lubbock discrepancy results were due to construction problems (bad compaction) in addition to environmental issues (very dry and windy conditions).

The following remarks are depicted in Figures 5.4 through 5.6:

a) From the regression analysis of these three sets of data, it can be seen that a poor correlation exists in the 700 to 1300 coulombs range. This point seems to agree well with Whiting's original research (Figure 5.4).
b) The original correlation between AASHTO T259 and T277 in the FHWA-81 report could not be used to accurately predict the permeability of concrete with a water cementitious range of 0.4 and 0.5 (Figures 5.5 and 5.6). However, the RCPT can serve as a general tool to compare the relative permeability of similar mixtures designed with similar constituents and admixtures.

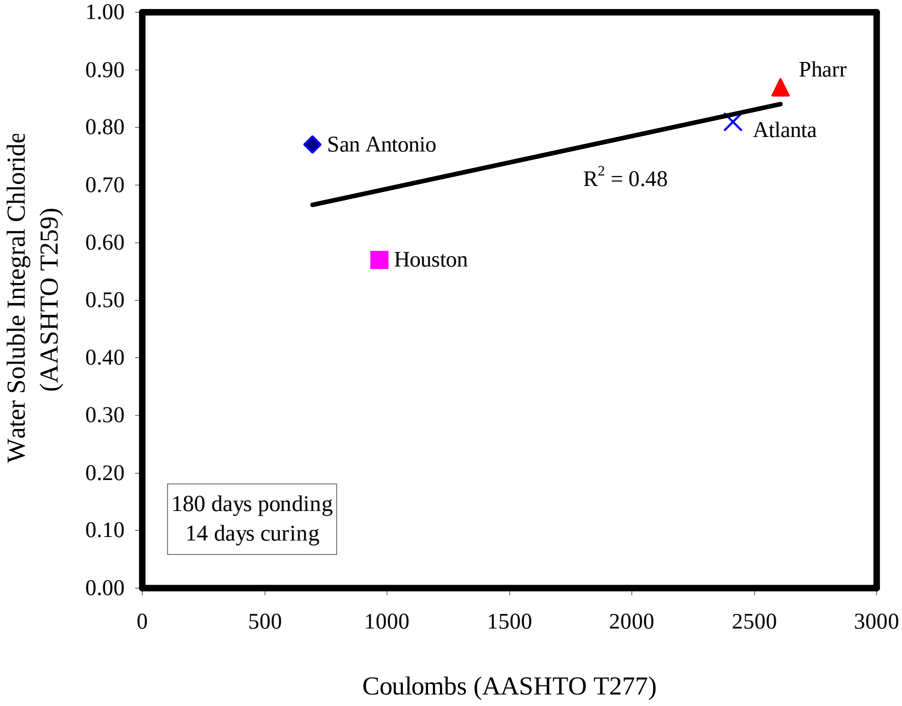


**Figure 5.4** Water Soluble Chloride Ion Content by Integral – Correlation Data

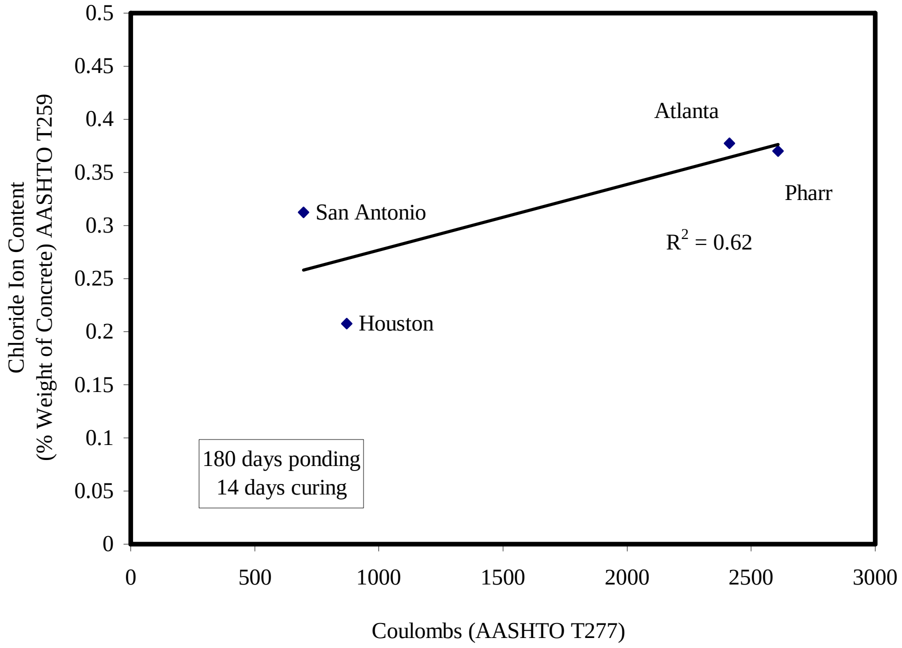


**Figure 5.5** Water Soluble Chloride Content by Percent Weight – Correlation Data

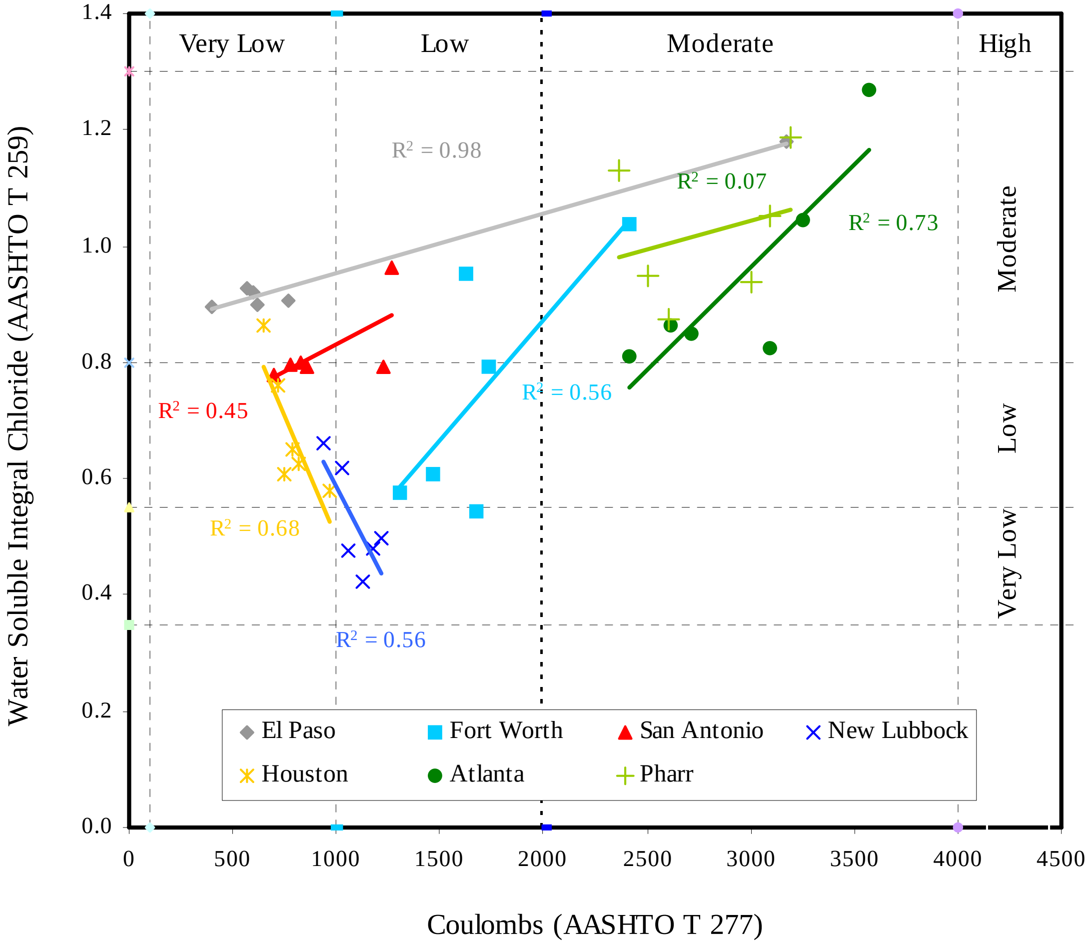


**Figure 5.6** Correlation of AASHTO T277 and AASHTO T259/260 for all Curing Durations

# CHAPTER VI
# SUMMARY OF RESULTS, CONCLUSIONS, AND RECOMMENDATIONS

A six-year research project was conducted to investigate the effects on concrete bridge deck durability from mixture design parameters, curing duration, and the age at which loads were applied. This report focuses on the effects of wet-mat curing duration and mix design parameters on chloride permeability of bridge deck concrete using AASHTO T259/260 ponding tests.

Permeability of concrete plays a major role in determining the service life of the bridge deck. For reinforced concrete bridge decks, a key form of deterioration is steel corrosion due to chloride ingress into concrete. This leads to subsequent deterioration in strength, serviceability, aesthetics, and overall durability of the bridge deck. Chloride ingress is primarily the result of de-icing chemicals used on bridge decks. This report discussed the mechanisms of corrosion and the factors that affect chloride ion permeability test methods.

## 6.1 Summary of Results

In summary, a comparison of permeability test results from all seven mix designs is shown in Figure 6.1. These mixes can be divided into three groups based on the chloride penetrability on the ponding test: those with lowest permeability (Fort Worth and New Lubbock), moderate permeability (El Paso, San Antonio, and Houston), and high permeability (Atlanta and Pharr).

Fort Worth and Lubbock (indicated as New Lubbock) mix low permeability results could be due to the following three reasons: 1) Fort Worth and New Lubbock were the only sites ponded for only 90 days, while the others sites were ponded for 180 days; 2) Fort Worth and New Lubbock were tested almost two years after casting, while the other five sites were tested 8 months after casting. As a result, the most hydration had occurred in Fort Worth and New Lubbock leading to the formation of more hydration products and thus a denser pore structure and lower permeability; and 3) the top surface was not abraded, thus providing extra protection to the concrete from the chloride ion penetration.

The second group includes El Paso, San Antonio, and Houston. As shown in Figure 6.1, permeability test results from these three sites lies between the results of the first and third groups. This can be intuitively expected since they were ponded for 180 days and had limestone coarse aggregate that displays better aggregate-cement paste bond.

The third group includes Pharr and Atlanta, where siliceous gravel was used as coarse aggregate. Permeability test results of this group were shown to be the highest. This may be due to the smooth surface texture of the siliceous gravel, which leads to the formation of a weak interface transition zone and to higher permeabilities.

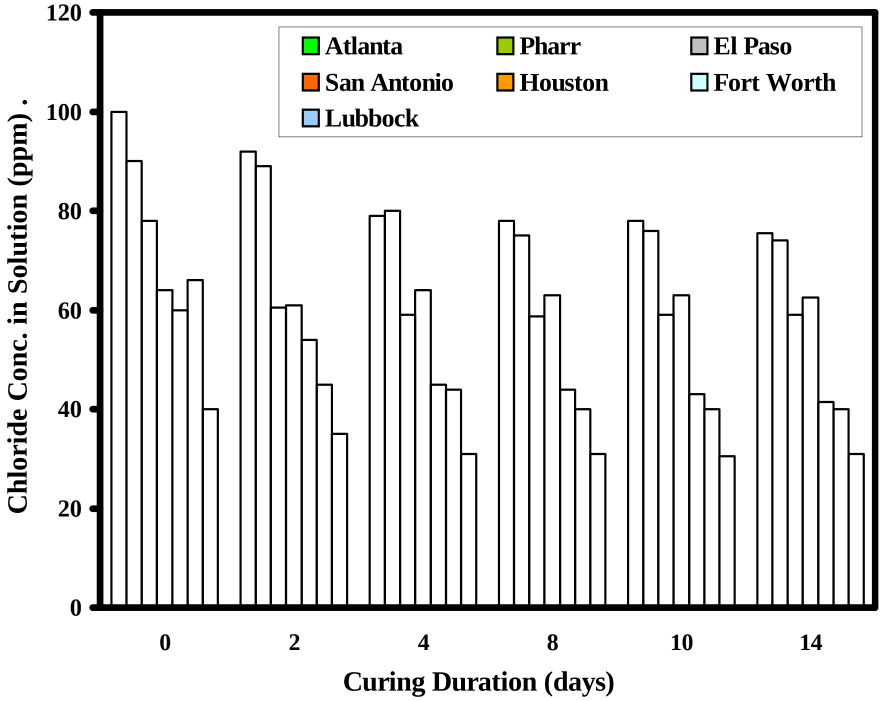


**Figure 6.1** Comparison of Permeability Results among the Seven Sites (layer 0)

Based on ponding test results, the effect of further curing on chloride permeability was negligible after 2 days of curing for the El Paso and San Antonio mixtures; after 4 days of curing for the New Lubbock, Atlanta, and Houston mixtures; and after 8 days of curing for the Pharr and Fort Worth mixtures. These finding are summarized in Table 6.1.

**Table 6.1** Summary of the Effect of Curing on Ponding and RCPT

| Site | Compressive Strength (psi) | No significant effect of curing after | |
|---|---|---|---|
| | | RCPT | Ponding |
| El Paso | 6117- 10 days of curing | 2 days | 2 days |
| San Antonio | 8156- 10 days of curing | 4 days | 2 days |
| New Lubbock | 6835- 10 days of curing | No clear trend | 4 days |
| Atlanta | 6315- 10 days of curing | 4 days | 4 days |
| Houston | 5762- 10 days of curing | No clear trend | 4 days |
| Pharr | 5250- 8 days of curing | No clear trend | 8 days |
| Fort Worth | 6464- 10 days of curing | 8 days | 8 days |

## 6.2 Conclusions

To investigate the effect of curing on chloride ion permeability of concrete, the ponding (AASHTO T259) test was chosen. Seven concrete mix designs used in different districts within Texas were tested in laboratory facilities located at Texas Tech University. At the beginning of the project, the AASHTO T260 test method was followed to determine the chloride content. However, several modifications were adopted. The first modification had the aim to reduce the time of testing. For this reason, a modified form of the AASHTO T260 test was developed. In a later phase of this research, a third method was adopted in which direct concentration inside solution specimens was measured using a chloride ion selective electrode. The second modification involved a change in the pattern of drilling. Pattern B was adopted because of the setup of the drilling machine used and to refine the result at the top surface of the sample (see Figure 2.8).

From the ponding results of the mixtures (Figures 4.13, 4.16, 4.19, 4.22, 4.25, 4.28, and 4.31), it was observed that chloride permeability decreased with increased curing duration. This is attributed to the fact that curing improves the hydration process, resulting in dense pore structures that lead to a decrease in concrete permeability.

Experimental results show that 2 days of curing were beneficial to the reduction of chloride permeability results for the El Paso and San Antonio mixtures. New Lubbock, Atlanta, and Houston results show that 4 days of curing were necessary to the reduction of chloride permeability, while the Fort Worth and Pharr sites show that 8 days of curing can be recommended (see Table 6.1).

Results from the El Paso site show that concrete permeability was relatively high compared to the other sites subjected to 180 days of ponding. Therefore, a different percent of cement had to be replaced by slag to receive satisfactory permeability results. In fact, TxDOT specifications currently recommend 30 to 35% of cement replacement when slag is used in a concrete mixture.

Fort Worth and New Lubbock sites, which contained 22% and 31% class F fly ash respectively, showed a low permeability (low and very low total integral chloride) due to the fact that the two sites were ponded for 90 days, the top surfaces were not abraded, and the samples were tested nearly 2 years after they were cast.

The Pharr and Atlanta sites, which contained siliceous gravel as coarse aggregate, show the highest permeability among all the sites. This could be due to the development of a weak aggregate-cement interface transition zone in concrete mixes containing siliceous gravel. This conclusion is asserted by getting moderate water soluble integral chloride and coulombs values (Table 5.3).

It was also found that permeability of concrete decreases as percentage of fly ash (either class F or class C) increases from around 20% to around 30%, and this is also shown in the comparison made between Fort Worth and New Lubbock mixtures (Figure 4.34) and the comparison made between San Antonio and Houston mixtures (Figure 4.38).

From the seven sites under study, it was shown that steel reinforcement can be put at a distance of 1.25” below the top surface because chloride concentration falls below the threshold chloride content at 1.25” for all of the samples at all of the sites. However, in reinforced concrete bridges where large amounts of concrete are used, thermal cracking in addition to plastic shrinkage is expected, and thus a higher cover thickness is recommended.

A comparison has been made between the ponding test (AASHTO T259/260) and the RCPT (AASHTO T277) based on the water soluble integral chloride content for all curing durations and on the percent weight of concrete. From this comparison, it can be concluded that there exists a correlation between AASHTO T259 and AASHTO T277, although the correlation is not perfect. This discrepancy arose from the fact that the integral chloride content resulted in a higher classification. It is also shown that the correlation using the percent weight of concrete ($R^2 = 0.62$) is higher than the correlation using the water soluble integral chloride ($R^2 = 0.48$).

From the comparison of results of these two tests, it was found that the classification of El Paso, New Lubbock, San Antonio, and Houston mixtures based on these two tests showed different trends, while trends were the same for Pharr, Atlanta, and Fort Worth mixes, which leads to the conclusion that RCPT may not represent the true long-term chloride penetration. This conclusion is asserted by a study by Walsh (et al. 1998), in which they show that only 1% of chloride ions pass through concrete during the RCPT. Also, the electrical conductivity from RCPT is mainly due to hydroxyl ion conductivity. Therefore, it may be better to consider an alternative test method that can provide a direct measurement of the chloride ion concentration rather than an indirect process where concrete conductivity is used. This is the main reason ponding is always recommended, although it has practical limitations such as length of test.

### 6.3 Recommendations for Future Research

The focus of this research project was to determine the concrete permeability using the ponding test (AASHTO T259/260), while an additional effort was made to study the correlation between the ponding test and the RCPT. Also, the test results carried out in this research project were from only seven mix designs at seven sites located across Texas. In future research, effort should be made to:

- Verify the results obtained in this research project by developing new concrete mixes with fewer variables (type of cement, aggregate type, percent voids air, percent cementitious materials, and materials sources).

- Determine the appropriate percentage of slag in the concrete mixture to obtain low permeability results and to avoid the discrepancy in classification between the ponding test and the RCPT shown in this research.
- Determine a well-verified correlation between AASHTO T259/260 and AASHTO T277, which requires 50 to 80 design mixes with different cementitious materials (fly ash, silica fume, and slag).

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