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CORRESION RESISTANCE AND SERVICE LIFE OF DRAINAGE CULVERTS

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16. Abstract

Laboratory/yard tests and a field survey were conducted to determine how chloride-induced corrosion limits the durability of reinforced concrete culvert pipe, and to revise as needed Florida D.O.T. guidelines for predicting durability. Laboratory tests exposed culvert pipe segments from two different manufacturers to cyclic and continuous saltwater ponding for up to 4.4 years. Yard tests were conducted with full size pipes exposed to saltwater for over 2 years. These tests revealed substantial chloride penetration and corrosion initiation during the exposure period. The results suggest that performance may be improved by the use of higher cementitious content concrete containing fly ash. Culverts exposed in the field to coastal seawater for estimated periods ranging from 27 to 43 years were examined and showed conditions ranging from mild corrosion to severe cracking, although all pipes appeared to be fully functional. Concrete cores of the field samples showed extensive chloride penetration. The results indicate that typical production culverts provide no exceptional protection against chloride penetration or initiation of corrosion, and that evaporative chloride concentration was important. It is proposed that an approach similar to that used by FDOT for estimating service regimes of marine substructure be applied to culvert pipe so that pipes in saltwater having above 2,000 ppm chloride, and having portions immediately above high tide, be treated for service life estimation the same as if exposed to seawater service. It is proposed to retain the present approach (using actual environmental chloride content) for estimating service life for continuing immersion, or environmental water chloride contents of less than 2,000 ppm.

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NOTICE

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The cooperation of the personnel of the Corrosion Laboratory of the Materials Office, Florida Department of Transportation in the field surveys is gratefully acknowledged.

EXECUTIVE SUMMARY

The main thrusts of this work were to obtain experimental and field information on the parameters, especially chloride exposure, that determine the durability of reinforced concrete culverts in Florida highway drainage applications, and to revise as required guidelines for predicting the durability of reinforced concrete drainage culverts. To address those thrusts, activities were conducted in four areas: 1-laboratory assessment of corrosion performance of reinforced concrete culverts specimens, 2-test yard assessment of corrosion performance of full size culvert segments, 3-evaluation of corrosion condition of culverts in the field, and 4-proposed revision of durability prediction guidelines.

Laboratory and yard tests of corrosion initiation and propagation were conducted over a period of several years with regular production culvert pipes from two different manufacturers designated by O (concrete made with unblended Type II cement) and F (concrete made with cement blended with fly ash). Laboratory exposure for 4.4 years to cyclic saltwater ponding resulted in initiation of sustained corrosion in all specimens of the F type, but in none of the O specimens. Laboratory exposure for 3.4 years to continuous simulated seawater ponding resulted in sustained corrosion initiation for only one of the F specimens, and none of the O specimens. The difference in corrosion behavior between O and F does not necessarily reflect any special corrosion resistance of the type O pipe materials, as the O specimens tested had significantly thicker average rebar concrete cover than those of the F specimen group. Corrosion rates in the laboratory specimens that showed sustained initiation were modest on average, but corrosion appeared to be localized. No external manifestations of corrosion (rust stains, cracks) were detected in the laboratory specimens. Chloride penetration from cyclic ponding in both types of materials was substantial. The results suggested that chloride binding was stronger in the F than in the O specimens. Apparent chloride diffusivity for both concretes (especially O) was higher than that commonly encountered in low permeability concrete, even though in both cases the water to cementitious ratio was very low. This behavior may be due in part to the modest cementitious content reported for both materials, and suggests that performance may be improved by the use of higher cementitious content concrete containing fly ash. Tentative estimates of critical chloride concentration in the specimens that underwent corrosion initiation yielded values somewhat larger than those normally reported for steel in concrete.

A yard exposure facility with full-size samples of O and F culverts was implemented and operated with saltwater for over 2 years. Results to date suggest that the steel has reached a condition of sustained active corrosion. No external manifestations of corrosion (rust stains, cracks) were detected yet in any of the yard-exposed culverts.

The culvert field survey sampled sites ranging from benign conditions to extremely aggressive service. As expected, culverts exposed in locations with low chloride, sulfate and moderate pH showed no indications of corrosion distress. Culverts exposed to coastal seawater for estimated periods ranging from 27 to 43 years showed conditions ranging from some rust staining at the exposed end of the culvert to extensive corrosion-related staining and severe cracking, although all culverts appeared to be fully functional. Concrete cores extracted from culverts in the saltwater locations showed extensive chloride penetration.

On implications to durability guidelines, it was noted that the laboratory and yard experiments have shown that corrosion initiation is possible in regular production culverts of at least one source after relatively short times in aggressive chloride environments. These observations are in agreement with the substantial chloride penetration encountered after relatively short times in the laboratory, and the extensive penetration recorded for the concrete of culverts in the field after several decades of exposure to salt water. It can be concluded that the concrete used for typical production culverts does not provide any exceptional protection against chloride penetration or initiation of corrosion. Thus, the length of the corrosion initiation stage should be projected as is commonly done for other reinforced concrete systems with similar concrete cover and average concrete quality. The laboratory results underscored the importance of evaporative concentration of chlorides in creating a highly aggressive environment. FDOT has in place guidelines used for estimating service regimes of marine substructure where evaporative concentration is important. It is proposed that a similar approach be applied to culvert pipes. Accordingly, it is proposed that pipes in saltwater having above 2,000 ppm chloride, and having portions immediately above high tide, be treated for service life estimation the same as if exposed to seawater service. It is also proposed that the actual environmental chloride content be used (as it is done currently) for calculating service life of pipes experiencing continuing or full immersion, or environmental water chloride contents of less than 2,000 ppm.

INTRODUCTION

Highway drainage culverts are a basic component of the State's transportation infrastructure. Durability of the culverts is an important cost issue because of the large number of units involved, and because replacement of a culvert already in service can require expensive demolition and reconstruction of associated roadwork. As a result, design service life requirements for highway culverts commonly approach 100 years. Such long durability requirement presents a difficult materials selection and performance prediction challenge. The challenge is made more difficult because attractive choices include materials such as aluminum or aluminum clad steel, which were introduced for this application only during the last few decades. More traditional materials, such as reinforced concrete, are subject to technology advancements with consequent need for reevaluation. Galvanized steel, with the longest service record, represents an important design alternative when a compromise between conservative design and overall cost needs to be considered.

The Florida DOT has periodically updated its durability design guidelines to reflect newly developing information on the performance of traditional and modern materials for culvert service. In addition, the FDOT Corrosion Research Laboratory and FDOT-sponsored work at the University of South Florida have produced data on the durability of culvert materials of direct relevance to Florida service conditions. Previous investigations of this type have resulted in FDOT Report No. 93-4A, "Drainage Culvert Service Life Performance and Estimation" [1]. This report, which includes the application software *Culvert Service Life Estimator*, summarizes the present FDOT guidelines for culvert materials selection with the best durability estimates based on presently available information. While the present guidelines serve the purpose of working design criteria, the considerations indicated earlier underscore the need to refine the design process and to continually develop up to date performance information.

The FDOT recognizes four classes of drainage culvert materials for which corrosion is likely: galvanized steel, aluminized steel, clad aluminum alloy, and reinforced concrete. The estimated durability of reinforced concrete exceeds that of the other materials in many service conditions. However, much uncertainty remains on the accuracy of the estimates in general, and specially for reinforced concrete. To take full advantage of the best material choice for each application, it is then particularly important to refine the durability estimate of reinforced concrete culverts.

Reinforced concrete deteriorates because of chemical degradation of the concrete itself (due to the presence of acidity and sulfates in the soil) but most notably because of corrosion of the reinforcement. The latter process involves first an initiation period, in which penetration of chloride ions or other aggressive species through the concrete cover and depassivation of the steel surface take place. A propagation stage follows in which the steel corrodes until the concrete cover is cracked [2]. Of critical importance is to determine the value and variability of the capacity to resist chloride ion penetration of present-day concrete formulations used in culverts. In addition, the extent of chloride buildup at the surface of the concrete was assumed but little was known about the actual values encountered in the field. Knowledge of those parameters was needed to develop design criteria better based on quantitative information than the present estimates. Improved methods for durability estimations are now becoming available as a result of several recent investigations on the progression of corrosion in reinforced concrete. These investigations include the massive research thrust of the recently completed Strategic Highway Research Program (SHRP) [1], the contributions of several European agencies, and work sponsored by FDOT on the durability of Florida bridges as well as the most recent laboratory work on Florida reinforced concrete culverts. In this investigation these recent developments were applied to validate, and if necessary update, guidelines for the use of reinforced concrete in Florida highway drainage applications.

The main thrusts of the work described below were to obtain experimental and field information on the parameters, especially chloride exposure, that determine the durability of reinforced concrete culverts in Florida highway drainage applications, and to revise as required guidelines for predicting the durability of reinforced concrete drainage culverts.

To address those thrusts, activities were conducted in four areas

1) Laboratory assessment of corrosion performance of reinforced concrete culverts specimens.

2) Test yard assessment of corrosion performance of full size culvert segments.

3) Evaluation of corrosion condition of culverts in the field.

4) Proposed revision of durability prediction guidelines

These activities and findings are addressed in the following sections.

AREA 1: LABORATORY ASSESSMENT OF CORROSION PERFORMANCE OF REINFORCED CONCRETE CULVERTS SPECIMENS.

Background

Current methods for service lifetime prediction of chloride exposed reinforced concrete structures assume the existence of a critical or threshold chloride concentration (C_{cr}) below which active corrosion is unlikely to occur. This threshold limit is generally expressed in terms of total chloride (i.e. 'bound' + 'free') where 'free chlorides' are dissolved in the pore water and 'bound chlorides' are either chemically or physically absorbed by the concrete. Any prediction of the length of the initiation stage (t_{cr}) is based on the speed of chloride accumulation at the rebar surface, and on computation of the time necessary to achieve a concentration equal to C_{cr}. A simplified forecast method consists of assuming planar geometry with a concrete surface that has a constant chloride concentration (C_s). Chloride ion transport through the concrete cover is then assumed to take place by homogeneous diffusion with time-invariant apparent diffusivity (D_{app}).

Two factors, which are of importance for service life prediction, are the concrete composition and the type of environment (e.g. intermittent or continuous contact with brackish water). The partial replacement of cement with fly ash, for instance, has been reported to reduce the threshold chloride concentration [3] but also, more markedly, the value of the apparent diffusion coefficient [4-6]. The environment has an important effect on the surface chloride concentrations, since at regions where wetting and drying cycles occurs, a progressive evaporative buildup of chlorides [6] takes place. Some researchers have suggested that C_{cr} is also affected by the environmental conditions, for example being lower in atmospherically exposed concrete than in permanently water saturated concrete [7,8].

In this portion of the investigation two types of concrete culverts were tested, one reported to be made with unblended Type II Portland cement (PC) and the other with PC blended with fly ash, designated by O and F respectively. The issue of environmental effect was addressed by using two types of ponding tests with chloride-rich water solutions. The specimens used for this laboratory investigation were cut from actual production pipes and had a range of concrete cover thicknesses, which offered an opportunity to examine the effect of dimensional variability on corrosion initiation times.

Experimental Procedure

Reinforced concrete drainage pipes 38 cm (15 in) inner diameter, from two different manufacturers and complying with ASTM C76, were used in this study. The concrete mix proportions and other data are given in Table 1. As reported by the manufacturers, the pipes of types O and F were made by the dry cast and by the packerhead methods, respectively. Type O pipes were demolded right after casting, kept indoors 24 h and then moved outdoors for air curing; type F pipes were also demolded right after casting, kept indoors overnight, and then moved outdoors for air curing. Both concretes were reported to have zero slump. The reinforcing cages of concrete pipes type O and F were made with 4.76 mm (3/16 in) and 6.35 mm (1/4 in) diameter steel wire, respectively. Both types of pipes had a wall thickness of 6.35

cm (2½ in). One pipe of each type was cut into 40.6 cm (16 in) long rings to obtain laboratory specimens. The rings were cut from the central bodies of the pipes avoiding end sections. Each ring was then cut into quadrants in order to get at least one longitudinal steel wire near the center of the quadrant. The resulting specimens (Figure 1) were arc-shaped, 41 cm (16 in) long and 36 cm (14 in) outer cord.

Small spots of the inner surface of the specimens obtained from both types of pipes were chipped off with a chisel and phenolphthalein solution was mist-sprayed over the newly broken surfaces. Then carbonation depths were measured at different points of the inner surface. The average carbonation depths were ~4 mm and ~ 1 mm for type F and type O specimens respectively. These tests were performed shortly before the beginning of the test exposures described below.

Samples of concrete from each culvert type were evaluated for porosity per ASTM C642. At least 4 specimens from each type were tested.

Pore water pH measurements were conducted using a procedure described by Sagüés et al. [9]. Samples of concrete from each culvert type were saw-cut in the form of arches ~ 15 cm wide along the perimeter direction and ~ 10 cm long in the direction of the longitudinal axis of the pipe. Holes were drilled on one of the faces of the arch. The holes have a diameter of 3.5 mm and 35 mm depth (See Figure 2). A plastic fixture with a rubber stopper kept the holes covered. The samples were placed and kept in a conditioning chamber at 100% relative humidity. To each hole 0.25 ml of distilled water was added. The amount of water in each hole was measured and made up periodically with distilled water for a period of 43 days until no further replenishing of water was required. The pH of the water was measured 17 days and 67 days after the level of water was stable.

Twelve corrosion test specimens (six from each pipe type) were prepared by attaching acrylic ponding dams to the concave side with a clear silicone adhesive. The dam footprint was 28 cm x 15 cm (11 in x 6 in). The remaining concave surface and cut edges of the specimens were coated with an epoxy sealer paint. Each specimen had a stainless steel screw tapped into the end of one longitudinal wire to serve as the working electrode connection. The wire cage was electrically continuous. Six specimens (3 of type O and 3 of type F), hereafter named “set A”, were exposed to cyclic ponding with 3% NaCl solution to allow for a wetting-evaporation cycle regime. Each exposure cycle consisted of 2 weeks ponding followed by two weeks drying. During each cycle a new ponding solution with the same concentration was used, filling the dam to a height of 38 mm (1.5 in). A second set of six specimens, hereafter named “set B”, were subjected to continuous ponding during 8 months with distilled water, after which the pond was emptied and filled with simulated seawater (ASTM D1141) with an ~20,000 ppm chloride concentration. Each specimen of set B was enclosed in a plastic bag, and distilled water was periodically sprayed inside in order to maintain a high humidity environment. After ~1,000 days of saltwater ponding the water level in the dams of set "B" had dropped by ~1/3, at which time distilled water was used to make up to the initial level. Pond water conductivity measurements taken at ~1,300 days showed values of ~ 0.05 ohm^{-1} cm^{-1}, consistent with the initial simulated seawater concentration, except for specimens F4B and

F5B for which conductivity was ~0.025 ohm^{-1} cm^{-1}. The ponds solution was replaced with fresh simulated seawater at that time for future continuing testing.

Each specimen was named with a 3-character code as exemplified by F1A, which stands for specimen number one of concrete pipe type F, used in intermittent ponding (set A). The laboratory conditions were ~ 60% R.H. and 22 ^{o} C ± 2 ^{o}C.

For determination of chloride concentration profiles, selected specimens were drilled at increasing depths from the concave surface and the powders were analyzed for total chloride ion content by acid digestion, according to the standard procedure FM 5-516 established by the Florida Department of Transportation (FDOT)[10].

Distributions of concrete cover depth (cc) for all twelve specimens were obtained by measuring the minimum distance between the specimen inner surface and each wire cross section exposed in the cut sides. Each specimen yielded about 12 cover depth values, which were then examined statistically.

The results presented here correspond to a total exposure time of ~1650 days for set A (55 cycles) and ~1300 days of continuous salt water ponding for set B. Continuing test exposures are in progress. Parameters monitored during the corresponding exposure times, included open circuit potential of the wire (E_{oc}), polarization resistance (R_p) and concrete resistivity (ρ).

For the electrochemical tests on each specimen, a saturated calomel electrode (SCE) was placed in one corner of the acrylic dam. A 7.6 cm (3 in) by 12.7 cm (5 in) activated titanium mesh [11] was temporarily placed at the bottom of the dam and used as the counter electrode. All electrochemical tests were conducted when the ponds were full. Conventional polarization resistance tests were conducted by varying the potential (starting from E_{oc}) in the negative direction, at a scan rate of 0.05 mV s^{-1}. Surface-normalized apparent polarization resistance values were obtained by multiplying the apparent polarization resistance ($R_{p,app}$) by the nominal specimen exposed metal area. This area was considered to be that of the reinforcing cage section defined by the ponding dam footprint. The values used were 237 cm^2 (36.8 in^2) for specimens type F and 178 cm^2 (27.6 in^2) for specimens type O. Conversion to nominal corrosion current densities (i_{corr}), after compensation for IR drop effects, was made by using a Stern-Geary constant B=26 mV [12].

Electrical resistance (R) was measured between the wire and the titanium mesh (during the wet cycle for set A) using a Model 400 Nilsson soil resistivity meter. R was then converted to a concrete resistivity value ρ, by multiplying it by a geometrical cell constant (C_k) which was determined using the heat transfer analogy for a row of tubes at equal depth in a semi-infinite solid [13]. A value of C_k was calculated for each specimen, taking into account the diameter of the steel wire, the average concrete cover depth and the size of the dam footprint. The typical cell constant calculated by this method was within 87-91% of the value calculated for two parallel plates.

The data of E_{oc}, i_{corr}, and ρ for set A presented in this paper are from measurements normally made near the end of each wet cycle. Measurements for set B were made approximately monthly.

Results

The average porosity for specimens type O was 11.2% (0.62 maximum-to-minimum range), while the average porosity for specimens type F was 13.5% (1.11% range). Values of pH were measured as 13.01 and 13.10 with ranges of 0.3 and 0.18 for specimens type O and Type F respectively. Both type O and F specimens had densities (fully wet) nearly equal to 2.3 g/cm^3, a value that was used as the base to calculate the volumetric chloride content following chemical analysis of powder samples.

Figures 3 (a) and 3 (b) show plots of the cumulative % of exposed wire cross sections in the cut edges of the specimens versus clear concrete cover thickness, for set A and set B, respectively. The average cover for type O specimens was generally greater than for type F. One exception was specimen O2A for which approximately 50% of the spots examined presented values of cc in the same range as specimens F1A and F3A for the same cumulative level. The lowest values of cc (about 12 mm (~ 0.5 in)) were found in specimens F2A, F4B and F5B. As expected, in both types of concrete pipes (O and F) the longitudinal reinforcement was closer to the inner surface of the pipe than the transversal (hoop) reinforcement. Figure 4 confirms that the lower values of cc were normally found in the spots corresponding to longitudinal wires, although in some cases, probably due to tight winding of the hoop wire, the opposite was observed. In Figure 5 the cumulative frequency distributions of concrete covers for specimens F1A and F2A are plotted again, but using different symbols for wire cross sections examined in different cut edges of the specimens. It is clear that the cover was not uniform around the pipe perimeter, showing some degree of eccentricity in the placement of the reinforcing cage within the walls of the concrete pipe.

The chloride concentration profiles at two different time periods for selected specimens of each pipe type are presented in Figures 6 and 7. The chloride profiles for F specimens showed a pronounced maximum a few mm inside the concrete surface, ahead of the carbonation front (located at 4 mm from the surface). Such an effect has been observed by other authors [2,14-16]. It has been proposed that in carbonated layers both the porosity [17] and the chloride binding ability [15,16] of the concrete are reduced, thus decreasing the total chloride content compared to the amount present in uncarbonated concrete exposed to the same medium. The O specimens had an apparent carbonation depth of only ~1 mm and did not show such a pronounced peak in the chloride concentration profile as in the F specimens. However, the concentration in the O specimens exposed to cyclic ponding showed a near plateau in the first ~20 mm from the surface, which cannot be explained at present. Nevertheless the O specimens showed lower chloride concentrations in the region ~6mm to ~25 mm from the concrete surface than those observed for the companion F specimens, which suggests that the O mix had less chloride binding capability than the F mix. This interpretation would agree with reports by other authors of greater chloride binding in fly ash versus unblended cement concretes [18,19].

Figure 8(a) shows the trends in open circuit potentials measured near the end of each wet cycle, for the specimens of set A. The initial potential for all of them was about –150 mV vs CSE. The E_{oc} values remained high (suggesting passive behavior) throughout the entire test period only for specimens O1A and O3A. The potential of specimen O2A experienced a drop to –360 mV vs CSE as early as about 50-90 days, but oscillated between low and high values afterwards. For specimens F2A F1A and F3A a more permanent potential drop occurred at about 120 days, 340 days, and 1000 days, respectively. The E_{oc} values for those specimens tended to attain, at the end of the test period, a roughly constant value near –400 mV vs CSE. The plots of E_{oc} versus exposure time for specimens of set B, given in Figure 8(b) show that for all the “O” specimens the potentials remained high throughout the 1300 days of exposure to simulated sea water. The E_{oc} values of the “F” specimens tended to be more negative than those of the "O" specimens but showed, for F5B and F6B, indications of passive behavior by stabilizing at a value of ~-200 mV vs CSE near the end of the exposure period. However, the potential of F4B drifted toward highly negative values near the end.

Figures 9 (a) and (b) show equipotential contour maps for specimens F1A and F2A, plotted on a suitably scaled plan view of the dam footprint. The potentials were measured placing a SCE sequentially over points of a grid (1/2 in centers) traced on the specimen’s inner surface. The measurements were made after 9 days drying during the 21st dry cycle (330 days). It can be observed that, although the maximum potential difference within each specimen does not exceed 75 mV, there are regions with potentials less than –300 mV vs SCE. The dashed lines in both figures represent the approximate locations of the wires under the ponding area. The data in Figure 5 suggest that for specimen F1A the part of the reinforcing cage on the right hand side of the dam area was closer to the specimen inner surface. Figure 9 (a) shows that it was precisely in that region where the potentials were more negative. A similar observation can be made for specimen F2A, but with the region with closest distance to the inner surface being on the left hand side of the dam area.

Apparent corrosion current densities (i_{corr}) for the specimens of sets A and B are presented, respectively, in Figures 10 (a) and (b) as a function of exposure time. In agreement with indications from the trends in E_{oc}, two specimens (O1A and O3A) in the cyclic ponding tests had very low apparent corrosion rates (~0.01 µA $cm^{-2)}$ suggesting that they were still under passive conditions at the end of the test period. For specimens O2A, F1A, F2A and F3A the time for the onset of active corrosion, inferred from the open circuit potential trends, seems to be consistent with the time when i_{corr} reached values several times greater than the low current density plateau of O1A and O3A. The highest sustained apparent corrosion current densities (F1A, F2A) were roughly between 0.1 µA cm^{-2} and 0.3 µA cm^{-2}, thus suggesting a moderate corrosion rate. For the continuously ponded set only one specimen (F4B, consistent with the open circuit potential trends) reached apparent corrosion current densities indicative of sustained active corrosion. In this case the highest values were also modest (1-2 µA cm^{-2}) but larger than for the cyclic ponding specimens.

Figures 11(a) and 11(b) show the evolution of concrete resistivity as a function of exposure time for set A and set B, respectively. For specimens of set A, the concrete in the type F culvert had initially resistivities four times higher than that of the type O culverts. The value of ρ seemed to increase with exposure time for both types of concrete and at the end of the test period, values in the order of 30-70 kΩ cm had been reached. For set B the resistivities were in the order of 30 kΩ cm for type O and 100 kΩ cm for type F, and did not change significantly throughout the exposure to simulated sea water.

Discussion

Corrosion initiation period

The chloride concentration profiles and the observed times to active corrosion initiation can be used to obtain estimates of key durability design parameters. Nominal values of the surface concentration C_s and the apparent diffusion coefficient D_{app} were obtained by fitting the concentration profile data to the solution of a one dimensional simple diffusion model with zero initial bulk concentration and constant surface concentration, represented by the following equation, in which x is the distance from the surface and t is the time:

$$C(x,t) = C_s\left(1 - \operatorname{erf}\left(\frac{x}{2\sqrt{D_{app}t}}\right)\right) \qquad (1)$$

The results for the *cyclic ponding tests* will be discussed first. For F specimens the analysis was carried out using only data for points beyond the 4 mm carbonation front and it was assumed, as a working approximation, that the position of constant concentration (x=0) was located 4 mm from the concrete surface. The values of C_s and D_{app} obtained by this method are presented in Table 2. Application of Eq. (1) to the data in Figures 6 and 7 involves numerous simplifications, as evidenced also by the visible departure of the concentration profiles from an ideal error-function shape. Nevertheless, some tentative observations may be made. The results of the analyses of the profiles obtained after ~760 and 1,650 days of exposure are comparable, suggesting that the system had reached a mature transport regime relatively early. For the F specimens, the surface concentrations estimates appear to be in keeping with those commonly obtained in ponding regimes of this type [19]. The values of D_{app} for the type F specimens ($1.4\ 10^{-8}cm^2s^{-1}$ - $2.9\ 10^{-8}cm^2s^{-1}$) are on the order of values typically obtained for somewhat permeable concretes [5,6,20]. This behavior may be due to the reported low cementitious factor of the concrete used (only 295 kg/m^3), which may be responsible also for the relatively high porosity (13.5%). The performance loss from the low cementitious content may have been only partially compensated by the beneficial pozzolanic replacement and the small water to cementitious ratio used.

Concentration profiles for specimens of type O were even broader (and D_{app} was consequently several times greater) than those for type F specimens. The reported concrete mix for type O specimens had greater cementitious content (~342 kg/m^3) and lower water to cementitious ratio than those reported for type F specimens. Those factors may have caused the lower observed porosity in mix O, but lesser binding capability than in type F (suggested earlier) may have prevailed in promoting faster diffusion in mix O. This interpretation must be considered only as speculative since many other factors (e.g. leaching into the pond water) could have differently affected the near-surface behavior of both O and F specimens.

Consequently, a quantitative evaluation of the relative binding ability of both materials will not be attempted with the presently available results.

The specimens showing the earliest indications of corrosion initiation during the test period were among those with the lowest minimum cover values within each group. The potential maps in Figure 9 show that for specimen F1A the more negative potentials are in the right hand side region of the grid. According to the distribution of cover depths (Figure 5) the longitudinal steel wire in that region was the closest to the inner surface, with a minimum cover depth cc_{min}=19 mm. Similarly for specimen F2A, the more negative potentials corresponded to the longitudinal wire in the left hand side of the dam area, and cc_{min} was found to be 16 mm.

To obtain estimates of the chloride threshold level for specimens F1A and O2A, their concentration profiles at 760 days were scaled to the observed times for initiation t_{cr} of corrosion activity (340 days for specimen F1A and either 80 days or >1650 days for specimen O2A) assuming that the value of x for any given concentration is proportional to the square root of time [22]. Such relationship stems from the simplifying assumptions for Eq.(1) and must be viewed only as a means to obtain a rough estimate. The values of C_{cr} were then determined by finding $C_{cr}=C(x=cc_{min}, t_{cr})$, interpolating as necessary or bounding if cc_{min} fell outside the scaled profile. A comparable analysis was conducted for F3A but using the profile obtained at 1,650 days and assuming (also for O2A) that x was the lowest cover value in the specimen. Based on the assumption that the chloride penetration pattern might be similar for specimens cut from the same concrete pipe and exposed to similar conditions, the profile for specimen F1A was also scaled to 120 days to estimate C_{cr} for specimen F2A. The results are presented in Table 3. The estimated C_{cr} ~<0.3 wt % binder for O2A under cyclic ponding was made by declaring that the end of the initiation period of specimen O2A took place around day 80 (first potential drop and increase in apparent corrosion current density). However, the depassivation in O2A was not permanent, as evidenced by the oscillating trend in E_{oc}, the eventual return to high potentials, and the low values of apparent corrosion current density (~0.05 $\mu A\ cm^{-2}$). These observations suggest that the end of the initiation stage is best defined as the moment of permanent passivity loss, and that approach will be used in the following. With that interpretation, the chloride profiles for the O specimens obtained at 760 days and 1650 days indicated that C_{cr} > 1.3 wt% binder for that culvert material combination. Overall, the C_{cr} estimates in Table 3 for the cyclic ponding tests show considerable variability but the lowest values are comparable with typically reported values for the concentration threshold in concrete [23].

For the *continuous ponding* tests in simulated seawater, the chloride concentration profiles in Figures 6 and 7 were obtained for specimens F5B and O4B after 390 days and for O6B and F6B after 1300 days respectively. Nominal values of C_s and D_{app} were obtained by fitting the data as indicated earlier (except that for O6B the computation program was allowed to identify a value of background concentration for best fit, ~0.12 kg/m^3), and they are presented in Table 2. The C_s values are markedly lower than those in the cyclic ponding regime, as expected since in these specimens there is no evaporative enrichment of the solution at the surface. Even so, particularly low chloride levels were measured in specimen O6B. Those lower values cannot be ascribed to dilution of its ponding water, which when tested had conductivity within the expected range, so this issue will need resolution during future continuation testing. The average values of D_{app} for the F and O specimens were similar, and significantly lower, respectively to those obtained in the cyclic ponding tests. However, the low concentrations in O6B may have introduced appreciable error in the determination of D_{app} for that specimen, which if disregarded would remove much of the difference between continuous and cyclic ponding diffusivity in Type O specimens.

During the entire test period there was sustained corrosion initiation in only one of the continuously ponded specimens, (F4B, t_{cr} ~600 to ~900 days). The C_{cr} estimate for this case is also reported in Table 3 and is comparable to the highest values obtained for the cyclic ponding tests. In this case as in the cyclic ponding specimens other than F1A and F2A, C_{cr} may have been overestimated by using in the calculation the lowest cover in the specimen instead of the cover for the actual corrosion initiation spot.

The wet concrete resistivity values reported in Figure 11 are significantly higher (especially for Type F) than those expected [21] for concretes having absolute values of D_{app} on the order of those reported in Table 2. In the Type F specimens, the carbonated layer on the surface of this specimen may have contributed to enhanced resistance between the steel and the ponded region. This possibility and alternative explanations to the higher than expected apparent concrete resistivity need to be explored in continuing work.

Corrosion Propagation Period

Only a fraction of the laboratory specimens tested have reached the propagation stage of corrosion, and none of the laboratory specimens have shown any evidence of corrosion-induced cracking or surface rust in the ponded region. Some rusting at points where wires intersect the cut sample edges has been observed, but cleanup of those spots showed only superficial rusting, and no change in the overall electrochemical response was detected after cleanup and epoxy patching. The nominal corrosion rates shown in Figure 10 are moderate even for specimens that had entered an apparently well-defined active corrosion period. However, the corrosion was very likely localized to small portions of the reinforcement assembly, as evidenced by the potential maps in Figure 9. Preliminary examination of the potential maps based on modeling of corrosion distribution in concrete [24] suggests that the area of active corrosion in those specimens was about one order of magnitude smaller than the total steel area of each specimen. Therefore (keeping in mind the numerous sources of error in corrosion rate estimation from polarization measurements in concrete [25-27]), corrosion current densities in the corroding spots of specimens F1A and F2A (with the highest apparent rates of the cyclically ponded specimens) may be estimated to be on the order of 1-3 $\mu A/cm^2$, or about 10 -30 μm/year. If corrosion rates of this magnitudes were sustained after corrosion initiation, localized cracking of the concrete cover could be expected after a few years of corrosion [28-29], but widespread damage may take decades to develop since depressed potentials in adjacent areas may considerably extend the length of the initiation stage. The apparent corrosion rate was higher in the one constant immersion specimen showing active behavior, but immersed conditions are known to reduce the incidence of corrosion-induced cracking, possibly because of less accumulation of solid corrosion products at the steel-concrete interface. Except for localized loss of reinforcement cross-section, the time required to observed corrosion-induced cracking could be very large under those conditions.

Overall Durability Considerations

The limited results available to date indicate that in very aggressive saltwater service (wet-dry cycling) localized corrosion in one type of regular production culvert pipe (F) can initiate in as little as a few months. In the other pipe type (O), sustained corrosion initiation did not take place even after 4 ½ years of exposure. However, this likely reflects the generally lower concrete cover in the inside of F specimens compared to that of the O specimens (Figure 3) rather than any intrinsic corrosion resistance of the type O pipe materials. Chloride buildup with time was substantial in both pipe types (indeed, free chloride penetration seems to be faster in Type O) and the estimated pore water pH in both type F and O materials is similar. Thus, corrosion initiation should be expected to occur during the next few years in the type O pipe as well.

In general, the results suggest that the concrete used for typical production culvert pipe does not provide any exceptionally good protection against chloride penetration or initiation of corrosion. This may be partly the result of moderate to low cementitious content in both concrete types. The use of higher cementitious content while keeping the same proportion of fly ash replacement as in the type F pipe should be explored. For forecasting purposes, the development of corrosion initiation over an entire culvert body may be anticipated from cover distribution curves such as those shown in Figures 3 to 5, plus some indication of the dependence of C_{cr} on potential. However, more extended testing with a mature corrosion development pattern would be needed to make an adequate quantitative forecast. In the documented cases, corrosion rates after initiation appear to be sufficient to locally compromise cover integrity after a few years after initiation, but development of widespread damage may take decades.

The results also suggest that continuous immersion in seawater represents a less severe corrosion initiation regime, based on the lower levels of effective surface concentration and on the observation of only one corrosion initiation event to date. The corrosion rate in that one case appears to be greater than for the cyclically ponded specimens but the consequences to integrity of the cover may be less severe. An extended test sequence may reveal more useful information to evaluate the long-term durability prognosis under this service condition.

AREA 2: TEST YARD ASSESSMENT OF CORROSION PERFORMANCE OF FULL SIZE CULVERT SEGMENTS.

Full size pipes (38 cm (15 inch) diameter, 3.7 m (12 ft.) long) of the same stock as those used for the laboratory corrosion experiments described in the previous section, were tested by exposing in an open yard location at the University of South Florida Campus. The pipes (2 type O and 2 type F) were placed horizontally and parallel to each other (See Figure 12). The pipe group was covered by a sandy fill mound with a clear cover of ~ 15 cm (~ 6 inch) over the top and with sloping sides on each of the two external culverts. The mound left both the front and back ends of the pipes uncovered. Acrylic dams were placed at the ends of each pipe to retain up to 10 cm (4 inch) of water. The ends of the pipes were left otherwise unobstructed. Stainless steel bolts were placed at each end embedded in the concrete and in contact with the culvert reinforcement. Potential of the reinforcement was monitored between the stainless steel bolt and a Copper/Copper Sulfate Electrode (CSE) in contact with the concrete at the end of the pipe before water was placed inside, or in the water afterwards. Polarization resistance measurements were performed following the same instrumental settings as in the laboratory tests and using the stainless steel bolt for the working electrode connection, the CSE as the reference electrode, and a 6 ft length of rebar or steel wire immersed in the water inside the pipe as the counter electrode.

The pipes were monitored on location exposed to weathering but without water in the dams for ~550 days, at the end of which tap water was placed in the dams for ~ 100 days. The tap water was then pumped out and replaced with a solution of 3.5 wt. % NaCl, replenished periodically. Steel potentials were monitored throughout the entire period reported here, which extends to 2 years after initiation of the saltwater regime. Polarization resistance measurements were conducted periodically starting shortly after initiation of the saltwater regime.

Figure 13 shows the potential measurements as function of time for the reinforcement in each one of the pipes. The potentials were initially all distinctly characteristic of passive steel, but drifted to more negative values during the first ~ 1-year conditioning period without water. By the end of that period potentials in the O culverts were still indicative of passive conditions but had reached ~-500 mV CSE in the F culverts. After introduction of fresh water potentials in both F culverts and one of the O culverts decayed by another ~100 mV. After saltwater introduction the potential in the F culverts decayed slightly more to reach an apparently stable value of ~-600 mV CSE. Potentials in both O culverts reached ~-400 mV CSE after ~2 years of introducing saltwater.

Polarization resistance measurements (solution resistance subtracted) are reported in Figure 14. The area of steel sampled by the measurements is uncertain since current distribution is likely to be quite uneven. Accordingly, only the total polarization resistance (expressed in ohms, not ohm-cm^2) is reported, but a very rough estimate indicates that the lowest polarization resistances observed correspond to an apparent corrosion current density on the order of 1 μA/cm^2. Such value is comparable to the highest values observed in the laboratory specimens. Although experimental scatter is large, Rp for the F culverts was clearly several times smaller than that for the O culverts, and the latter appears to have a decreasing trend with time. To date, there are no externally visible symptoms of corrosion damage in any of the 4 culverts.

The evidence from the yard culvert test suggests that corrosion initiation has taken place after a few years of exposure to aggressive conditions for at least culvert type F, a finding consistent with the general findings of the laboratory specimens. The early indications of corrosion development in the O culverts need confirmation by observation of longer term trends. An objective in setting up the test facility was to permit evaluation over extended periods of time. The assembly remains in place and it is planned to maintain it indefinitely as resources permit.

AREA 3: EVALUATION OF CORROSION CONDITION OF CULVERTS IN THE FIELD.

Procedures and results

Culverts in service for 12 years to 43 years at 8 different locations (Table 4, coded 01 to 08) were examined to assess the effect of environmental conditions on service life. These locations had levels of environmental aggressiveness ranging from inland stormwater exposure to direct contact with seawater.

A sketch with dimensions and exposure conditions of each culvert is given in Figures 15 to 22, and a summary description of each exposure condition is given in Table 4 along with a listing of the key observations indicated in the following. Structure age was always subject to uncertainty but whenever possible it was estimated from FDOT Project number designations for the structures deemed to be associated with the culvert. The structures were visually inspected to determine overall appearance including evidence of corrosion, and the result was used to assign a rating designation ranging from "Very Good" to "Poor" to each culvert examined. Samples of soil around the culvert (usually near the crown) and water if available were analyzed for pH, Chloride and Sulfate content, and resistivity following standard procedures for soil and water analysis. Table 4 also notes the corrosion condition of steel present in cores that were extracted as indicated next.

The resistivity of the exposed concrete in the culverts was measured in-situ using a CNS concrete resistivity probe. Concrete cores were extracted from the crown and invert of each culvert at all locations except 08 (crown only). The spots sampled depended on overall accessibility. Figures 15 to 22 are schematic representations of each site arrangement and core extraction locations. Whenever clearing the soil over the crown was needed to drill out the cores, the condition of the exposed concrete was also taken into consideration for the visual appearance listing in Table 4.

The cores were examined for rebar position and concrete cover depth, evidence of rebar corrosion, and measured and weighed to estimate the unit weight of the concrete. Selected cores were sliced and the concrete slices were analyzed for chloride content as function of distance from the surface. Other cores were exposed in a 100% RH chamber until constant weight was reached, and then the electric resistivity of the concrete ("wet resistivity") was measured. Concrete core code designations and principal analysis results are given in Table 5. Core dimensions, slicing schemes, and chloride profiles are shown in Figures 23 to 29.

Discussion of results

As expected, culverts exposed for 17 years at locations 01 to 03 were in excellent condition. Those locations have benign environments with negligible chloride and sulfate and near neutral pH for which the present FDOT Drainage Culvert Service Life Estimator (DCSE) projects service life on the order of 100 years for the pipe sizes examined. The same can be said for Location 08, where conditions were also benign although periods of high moisture may be expected to be more prevalent than in locations 01 to 03. Concrete cores extracted from these locations showed virtually no chloride contamination, consistent with the observed environmental conditions.

As shown by the chemical analysis information in Table 4, the water in locations 04 to 07 had a composition typical of seawater (with perhaps some minor dilution), and the soil next to the culvert showed significant seawater contamination in three of those locations. These extremely aggressive conditions are consistent with the evidence of pronounced corrosion seen at 3 of those sites. As expected, the sites where the Crown was below high tide (thus less likely to experience evaporative chloride concentration) have the best overall appearance and less evidence of rust in the rebar inside cores. The two sites with Crown above high tide had the most evidence of damage. Although all culverts examined appeared to be still fully functional, the condition at location 07 (estimated age 43 years) appears to be indicative of a structure approaching the end stages of its service life (See Figures 30 and 31).

All the the cores extracted from locations 04-07 show high chloride concentrations (the highest was almost 18 kg/m^3 for a core extracted from the Crown in location 04). Even after allowing for a few kg/m^3 of bound chloride at the surface of the concrete [14-16], concentrations of such magnitude indicate that the concrete pores contain solution with a chloride content several times greater than that of seawater (and approaching saturation), most probably because of evaporative enrichment. In 3 of the 4 cases the chloride concentration profiles were nearly flat across the core, indicating that the characteristic transport distance $(\sim(D\ t)^{1/2}$, where D is the apparent diffusivity and t is the time) was on the order or larger than the culvert wall thickness. Since the average estimated age of these structures was about 35 years, the results are consistent with an apparent chloride diffusivity on the order of or greater than $2\ 10^{-8}\ cm^2/sec$. With the possible exception of location 6, the wet resistivity measurements produced values that are generally consistent with this estimate of the chloride diffusivity. This value is comparable to the values encountered in medium quality concrete exposed to moist conditions [21]. The results then support the conclusion that the concrete used for regular production culverts does not provide any exceptional protection against chloride ingress. Based on the observation of still functional structures after several decades of service, corrosion propagation in the saltwater locations appears to be moderately slow, also consistent with the laboratory observations.

AREA 4: PROPOSED REVISION OF DURABILITY PREDICTION GUIDELINES

Chapter 5 of FDOT Report No. 93-4A “Drainage Culvert Service Life Performance and Estimation” (Ref. 1) addresses durability of reinforced concrete culverts. The equation presently used to estimate service life (SL) in years is:

$$SL = 10^{3} \bullet 1.107^{Cc} \bullet Cc^{0.717} \bullet Dc^{1.22} \bullet (K+1)^{-0.37} \bullet W^{-0.631} - 4.22 \bullet 10^{10} \bullet pH^{-14.1} - 2.94 \bullet 10^{-3} \bullet S + 4.41 \qquad (2)$$

where S = Environmental sulfate content in ppm
Cc = Sacks of concrete (94 lb each) per cubic yard
Dc = Concrete cover in inches
K = Environmental chloride concentration in ppm
W = Total percentage of mix water by volume

Report No. 93-4A illustrates that for a typical 36-inch diameter culvert pipe with concrete cover Dc = 1.05, concrete with Cc = 6 bags and W = 9%, then SL ~110 years when K and S are small (e.g. < 1,000 ppm) and pH is neutral to basic (e.g. > 6). As K or S increase the value of SL decreases smoothly, reaching values on the order of 40 years when typical seawater conditions (K ~ 20,000 ppm and S ~2,500 ppm) are reached. Figure 32 (reproduced from Figure 5-1 in Report No. 93-4A) summarizes the overall SL predictions for concrete for the case of S = 1500 ppm.

The dependence on chloride concentration in Equation (2) was derived from the former work by Strattful [30] and by Sagüés [31], the latter based on the behavior expected from chloride diffusional behavior as described by Eq.(1). In Ref. [31] it was assumed tentatively that surface concentration on the concrete would on first approximation be proportional to the environmental chloride content, but caution was indicated concerning the effects of evaporative concentration. The evidence from the present work (contrast between the laboratory specimens with cyclical and continuous ponding, high chloride enrichment in the field culverts) underscores the importance of evaporative concentration in determining surface chloride content.

Because evaporative enrichment is limited to the solubility limit of chloride salts in water, evaporation plays an equalizing role in the surface chloride content in concrete, which is likely to be the same whether the external water contains 25,000 ppm Cl^- or only a fraction of that. An example of strong chloride enrichment from moderate water chloride content can be found in the substructure of the Rafael Urdaneta bridge in the tropical environment of Lake Maracaibo, Venezuela. The lake water has only ~3,000 ppm chloride but high concrete surface chloride contents (e.g. in excess of 2% of the cement content) have been recorded [33]. This situation is recognized by the FDOT guidelines used to designate extremely aggressive environments for the substructure of marine bridges [32], for which a 2,000 ppm Cl^- defining criterion is used. Both the laboratory and field portions of the present work have shown that concrete in regular production culverts does not behave very differently, from the standpoint of chloride buildup and transport, from medium quality concrete used in some marine bridge substructures [5,6, 9,20]. Therefore, it is proposed that a similar defining criterion of 2,000 ppm Cl^- be used to conservatively designate environments that will cause severe chloride enrichment at the culvert pipe surface when evaporative conditions are present. It is further proposed that for those cases the service life be estimated by applying Eq.(2) for the case of undiluted seawater of the highest concentration considered in FDOT Report No. 93-4A (25,000 ppm Cl^-), because the SL forecast for neutral seawater in that case (~40 years for a 36-inch pipe) agrees reasonably with the behavior observed in the field survey and also consistent with the general expectations from the laboratory results. No changes are recommended for estimating the effect of the other variables in Eq.(2), as not enough information has been developed to that effect in the present investigation.

For practical application, it is proposed that evaporative conditions be assumed whenever the end of the culvert pipe is exposed in tidal regions with the high tide line below the crown of the culvert. For those cases, SL is proposed to be determined by application of Eq.(2) with K = 25,000 ppm if the actual environmental chloride (Ka) exceeds 2,000 ppm, and with K = Ka when Ka <2,000 ppm. For applications where the high tide rises daily above the culvert Crown, or when there is continuing immersion of the culvert, it is proposed that K= Ka be used.

CONCLUSIONS

1. Laboratory and yard tests of corrosion initiation and propagation were conducted over a period of several years with regular production culvert pipes from 2 different manufacturers designated by O (concrete made with unblended cement) and F (concrete made with cement blended with fly ash). Laboratory exposure for 4.4 years to cyclic saltwater ponding resulted in initiation of sustained corrosion in all three F specimens at times ti ranging from ~ 0.3 to ~3 years, but in none of the three O samples. Laboratory exposure for 3.4 years to continuous simulated seawater ponding resulted in sustained corrosion initiation for only one of three F specimens at ti ~ 2.6 years, but no initiation for the other two F or three O specimens tested.

2. The difference in corrosion initiation behavior between O and F does not necessarily reflect any special corrosion resistance of the type O pipe materials, as the O specimens tested had significantly thicker average rebar concrete cover than those of the F specimen group.

3. Surface-averaged corrosion rates measured in the laboratory specimens that showed sustained corrosion initiation were on the order of 1 µm/year to 10 µm/year. Potential map measurements suggest localization of corrosion with corrosion rates about one order of magnitude higher than average rates. No external manifestations of corrosion (rust stains, cracks) were detected to date in the ponding regions of any of the specimens from either supplier.

4. Chloride concentration profiles for the laboratory specimens were used to estimate that the chloride diffusivity for O and F concrete was on the order of $\sim 10^{-7}$ $cm^2 sec^{-1}$ and $\sim 2 \bullet 10^{-8}$ $cm^2 sec^{-1}$ respectively. These diffusivity values are higher than commonly encountered in low permeability concrete, even though in both cases the water to cementitious ratio was very low. This behavior may be due in part to the modest cementitious content reported for both materials, and suggests that performance may be improved by the use of higher cementitious content concrete containing fly ash.

5. The critical chloride concentration for F was estimated to be between ~<0.8% and ~1.8% of the total cementitious content, a range of values somewhat larger than those normally reported for steel in concrete. The critical concentration for O was estimated to be > ~1.6% of the total cementitious content. These estimates are subject to considerable uncertainty and may be revised in continuing testing, which should also explore unresolved issues on chloride profile complexities and interpretation of concrete resistivity results.

6. Yard exposure in duplicate of full-size samples of O and F culverts was conducted by preconditioning without chloride contamination for ~ 2 years and then by partially filling the pipe with saltwater for ~2.2 years. Steel in O and F culverts reached long-term steady potentials of ~-300 mV and ~-600 mV (CSE) respectively. Apparent polarization resistance of the steel in the O culverts was several times greater than that in the F culverts. While interpretation of these results is not straightforward, these symptoms suggest that the steel in the F culverts had reached a condition of sustained active corrosion during the test period. No external

manifestations of corrosion (rust stains, cracks) were detected to date in any of the yard-exposed culverts.

7. The culvert field survey sampled sites ranging from benign conditions to extremely aggressive service. As expected, culverts exposed for 17 years in an inland location with neutral soil and moderate resistivity were in very good condition, had negligible concrete chloride concentrations and reinforcement found in cores had no indications of corrosion. Culverts exposed to saltwater (~19,000 ppm Cl^- and ~3,000 ppm $SO4^=$) for periods ranging from 28 to 43 years showed conditions ranging from good (some rust staining at the exposed end of the culvert) to poor (extensive corrosion-related staining and severe cracking), although in all those instances the culverts appeared to be fully functional. Concrete cores extracted from culverts in the estuary locations showed substantial chloride penetration; in several instances the chloride concentration exceeded 10 kg m^{-3} through the entire pipe wall.

8. The laboratory experiments have shown that corrosion initiation is possible in regular production culverts of at least one source after relatively short times in aggressive chloride environments. Measurements in the yard culverts suggest similar behavior. These observations are in agreement with the substantial chloride penetration encountered after relatively short times in the laboratory, even in concrete with very low w/c, and the extensive penetration recorded for the concrete of culverts in the field after several decades of exposure to salt water. It can be concluded that the concrete used for typical production culverts does not provide any exceptionally high protection against chloride penetration or initiation of corrosion. Thus, the length of the corrosion initiation stage should be projected as is commonly done for other reinforced concrete systems with similar concrete cover and average concrete quality.

9. Only a fraction of the laboratory specimens tested had reached the propagation stage of corrosion. For those, corrosion proceeded at a moderate spatially averaged pace but tended to be localized. Projections from the limited evidence available suggest that widespread corrosion-induced damage would need decades to develop. The evidence from the field is also consistent with an overall service life of several decades in highly aggressive chloride environments. These findings are consistent with the service life values (e.g. 41 years for 36 inch pipe in pH 6-9, 1,500 ppm Sulfate, 25,000 ppm chloride) estimated for a representative seawater service with the present FDOT (Report 93-4A) culvert design guidelines.

10. The laboratory results, overall knowledge of corrosion initiation in concrete, and the field observations all suggest that full immersion in saltwater tends to create a more benign environment than exposure with evaporative concentration of chlorides. The overall findings suggest using an approach comparable to that of FDOT guidelines for estimating service regimes in marine substructure. Thus it is proposed that durability of pipes in service in saltwater having both >2,000 ppm chloride and portions immediately above high tide be estimated with the procedures in FDOT Report 93-4A, but treated as if exposed to a nominal 25,000 ppm seawater service. For pipes experiencing continuing immersion, or environmental water chloride contents of less than 2,000 ppm, it is proposed that the current method of calculation (using actual environmental chloride content) be retained.

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TABLE 1
Mix proportions (from information reported by each manufacturer)

Mix ref.	Cement	Cement content (C)	Fly ash content (FA)	Limestone Coarse Aggregate	Sand Fine Aggregate	w/(C+FA)
		(Kg m^{-3})	(Kg m^{-3})	(Kg m^{-3})	(Kg m^{-3})	
O	Type II PC	342†	0	730†	1100†	0.33
F	PC* / Fly Ash**	236	59	997	999	0.37

*Expected to be Type II **Expected to be Type F
†Calculated assuming density = 2,300 kg/m^3

TABLE 2

Diffusion parameters obtained by regression using Eq. (1) and simplifying assumptions.

Environment type	Specimen	Exposure t	C_s	D_{app}
		(days)	(Kg m^{-3})	(10^{-8} cm^2 s^{-1})
Set A Cyclic Ponding	F1A	756	14	2.9
	F3A	1650	14	1.4
	O2A	756	7	16
	O1A	1650	6	7
Set B Continuous Ponding	F5B	390	5	2.3
	F6B	1300	7	1.8
	O4B	390	2.5	5.3
	O6B	1300	1.3	0.9

TABLE 3

Estimated chloride threshold concentrations based on simplified analysis

Specimen	t_{cr}	C_{cr}	
	(days)	(Kg m^{-3})	% binder content
F1A	340	2.5	0.85
F2A	120	<2.4	<0.8
F3A†	1000	5	1.8
O2A†	(80*) >1650	(<1.1*) >6	(<0.3*) >1.6
F4B†	600-900	4.7-5.3	1.6-1.8

* Active condition not sustained
† Concrete cover assumed to be lowest in specimen

Table 4 - Part 1. Summary of Field Evaluations

Site		Description	Age (yr)	Size (in)	Soil				Water				Overall Culvert Appearance[3]
					pH	Cl^- (ppm)	$SO_4^=$ (ppm)	ρ (Ω-cm)	pH	Cl^- (ppm)	$SO_4^=$ (ppm	ρ (Ω-cm)	
01	5901 NE Waldo Rd (NB), Gainesville FL	Inland	17	24	7.53[1] 7.59	20[1] 10	306[1] 99	5500[1] 13000	NA	NA	NA	NA	VG
02	US 301(NB) 0.7 mi. N of US 301 and NE Waldo intersection. Gainesville FL	Inland	17	24	7.32[1] 7.00	26[1] 17	132[1] 0	7800[1] 11200	NA	NA	NA	NA	VG
03	1907 Dale Earnhart Rd. (NB). Gainesville FL	Inland	17	30	7.64[1] 6.84	1.2[1] 24	24[1] 6[2]	3200[1] 1300	NA	NA	NA	NA	VG
04	SR-666 (SB) NW side of Tom Stuart Causeway. Culvert outflows into Boca Ciega. (15100-3150)[4] Madeira Beach FL	Draining out of seawall. Crown below high tide.	39	18	8.47	2190	420	190	6.82	19200	3100	23	G
05	End of 127th Ave. E side of Gulf Blvd. (15100-3509)[4] Madeira Beach FL	Draining out of seawall. Crown below high tide.	28	36	8.71	331	96	1600 40800[2]	7.6	18100	3100	23	VG
06	End of Tilden St.- W side of Bayshore Blvd. (15070-3516)[4] Dunedin FL	Draining into tidal beach. Crown above high tide.	42	24	7.70	54[1] 9.51	0	9891[2]	8.09	18453	3400	22	F-G
07	End of San Jose St. - W side of Bayshore Blvd. Dunedin FL	Draining into tidal beach. Crown above high tide.	43	30	7.92 7.72	345[1] 59.4	140[1] 0	125	7.83	18656	3400	23	P
08	SR 580 (SB) - NW side of the Safety Harbor Bridge.(15050-3511)[4] Oldsmar FL	Draining into fresh water canal	12	36	7.66	48[1] 0	0 0	2900	7.9	44 353	22 61	2000	VG

Notes.

[1] Inside soil sample (unless indicated otherwise, values are for outside soil samples)

[2] In-situ resistivity

[3] VG-G-F-P: Very good, good, fair, poor. See additional notes on structure condition in Table 4, Part 2.

[4] FDOT Project Number indicated as available

Table 4 - Part 2. Notes on Structure Condition.

Structure	Notes
01 to 03:	No evidence of corrosion related distress anywhere in culvert.
04:	When excavating to drill core on Crown, cracks (likely not corrosion related) were observed on concrete surface. There was some rust staining on exit rim of culvert.
05:	Rust staining observed on exit rim of culvert (probably connected with physical impact). Upon excavation to drill core on Crown, concrete surface was found to be absent of any distress.
06	Pipe showing conspicuous rust staining inside near exit, but no signs of significant distress.
07	Conspicuous cracking and rusting of external portion of pipe crown and sides.
08	Upon excavation to drill core on Crown, concrete surface was found to be absent of any distress. No evidence of distress seen elsewhere.

Table 5. Concrete core properties

Loc (#)	Core (#)	Date Extracted	Part of Culvert	Outside Cond.	Diam. (cm)	Length (cm)	Weight (gr)	Nominal Density (gr/cm^3)	Concrete Field Resist. (KΩ-cm)	Core Wet Resist. (KΩ-cm)	Notes	Condition of reinforcement in core
01	0101	07/28/99	crown	soil	4.8	8.0	343	2.4	66	23.6		NRP
01	0102	07/28/99	invert	soil	4.8	8.0	329	2.3	62			NRP
02	0201	07/28/99	crown	soil	4.8	8.0	325	2.2	33			NRP
02	0202	07/28/99	invert	soil	4.8	7.0	290	2.3	45	13.9		no steel
03	0301	07/28/99	crown	soil	4.8	8.9	378	2.3	43	10.8		NRP
03	0302	07/28/99	invert	soil	4.8	9.9	391	2.2	40			NRP
04	0401	09/09/99	crown	soil	4.5	6.1	246	2.5	43			NRP
04	0402	09/09/99	crown	soil	4.5	6.1	246	2.5	43	12.5		no steel
04	0403	09/09/99	crown	water	4.5	6.1	293	3.0	43			VSRP
04	0404	09/09/99	invert	water	-	-	86	-	43		cracked[1]	SRP
04	0405	09/09/99	crown	water	4.5	6.1	256	2.6	43		cracked	no steel
05	0501	09/09/99	crown	soil	4.8	10.2	353	1.9	26	16.3	cracked	NRP
05	0502	09/09/99	crown	water	4.8	10.6	456	2.4	26		cracked	NRP
06	0601	02/23/00	crown	air	4.8	7.6	322	2.3	320	27.5		no steel
06	0602	02/23/00	invert	water	4.8	8.0	344	2.4	35		cracked	R
07	0701	02/23/00	crown	air	4.8	9.7	419	2.4	60	8.2	cracked	R
07	0702	02/23/00	invert	water	4.8	7.3	300	2.3	16			no steel
07	0703	02/23/00	crown	air	4.8	9.7	411	2.3	30		cracked	no steel
08	0801	02/23/00	crown	soil	4.8	13.1	548	2.3	60			NRP
08	0802	02/23/00	crown	soil	4.8	13.4	560	2.3	76	16.1		NRP

NRP-VSRP-SRP-R: No rust present, very slight rust present, slight rust present, rusted.

[1] Core partially recovered

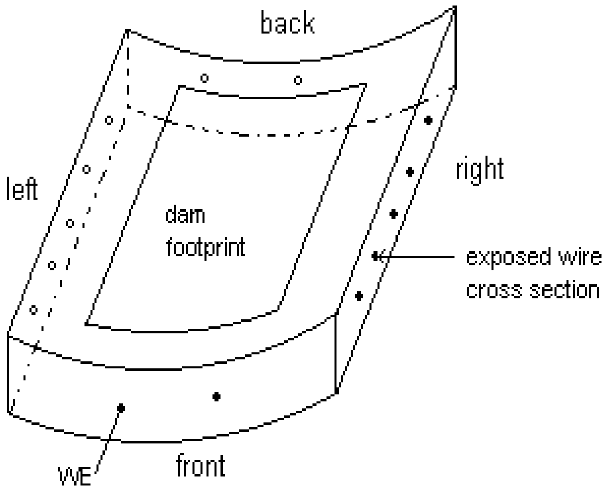


Figure 1. Sketch of the specimens used. The cut edge with the connection for the working electrode (WE) was considered to be the front of the specimen.

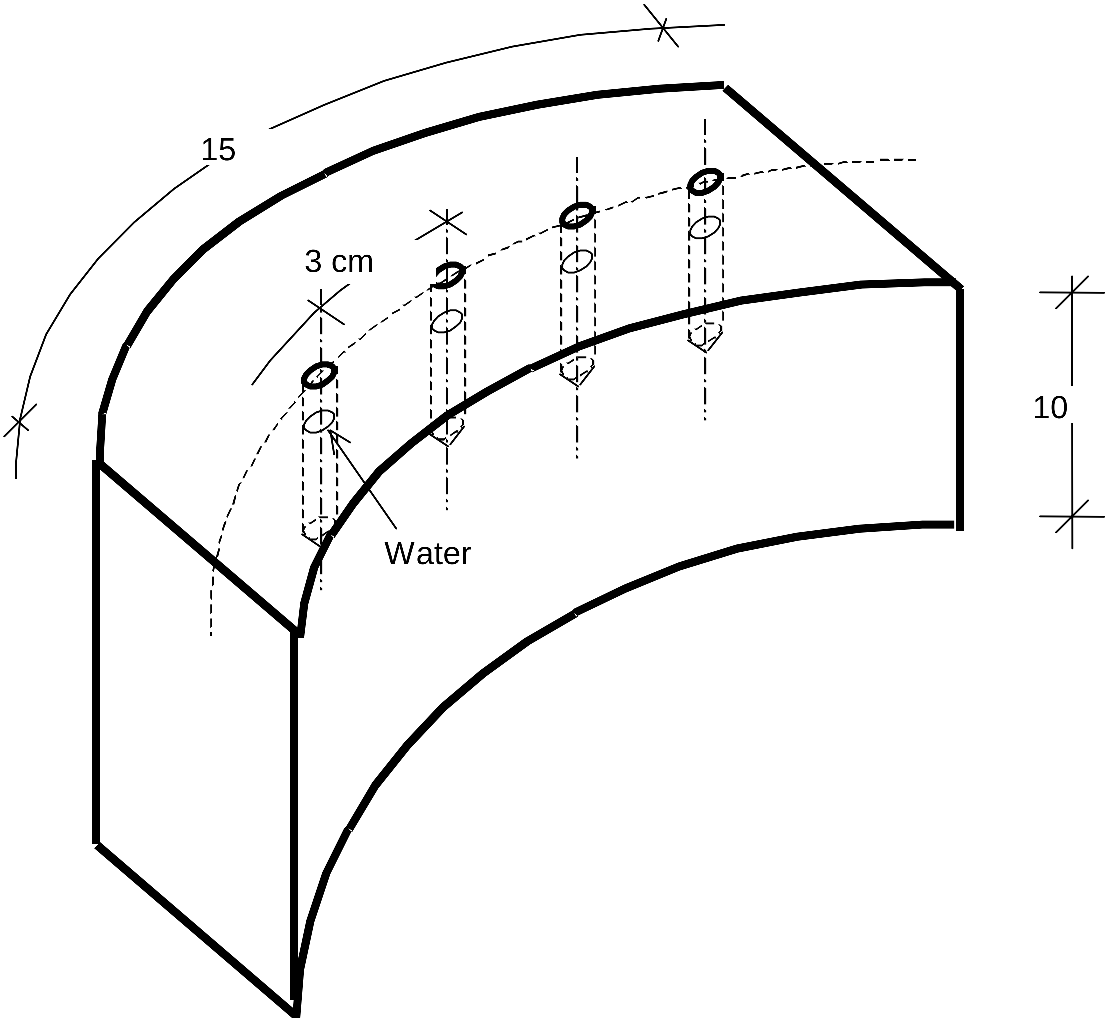


Figure 2. Concrete Culvert Segment for Pore Water pH Measurements.
Note: Plastic toppers are not represented in this

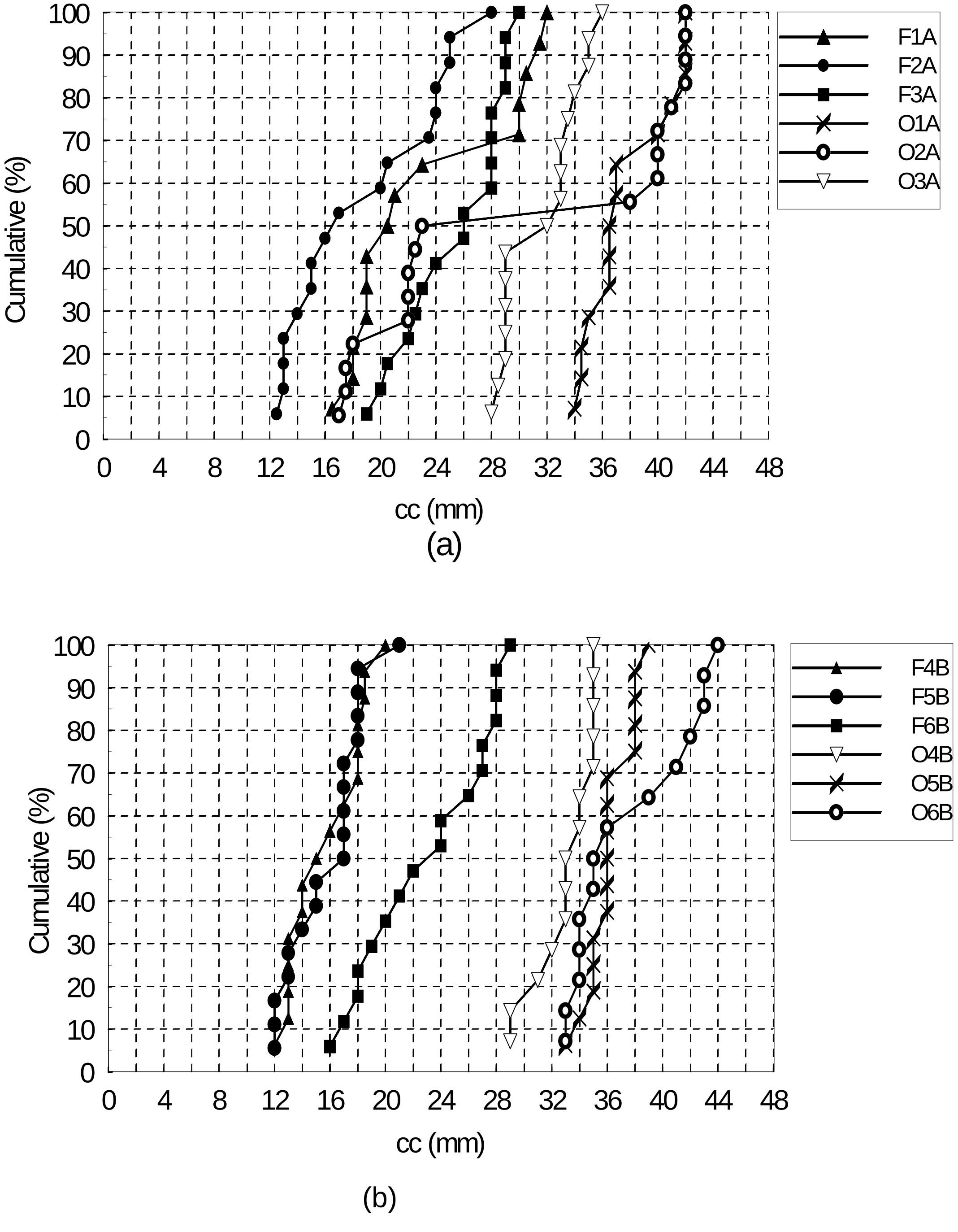


Figure 3. Cumulative percentage of steel wire cross sections examined in the cut edges *vs* concrete cover thickness (cc) for specimens corresponding to : a) set A and b) set B.

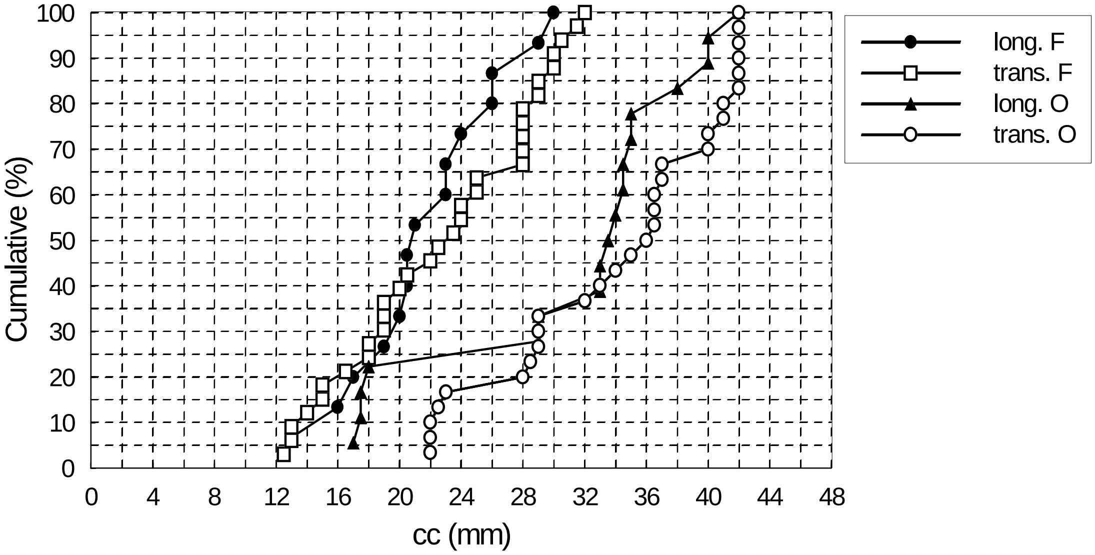


Figure 4. Distribution of concrete cover with cc values for cross sections of longitudinal and transversal wires plotted separately. Data for all six specimens of a) set A and b) set B.

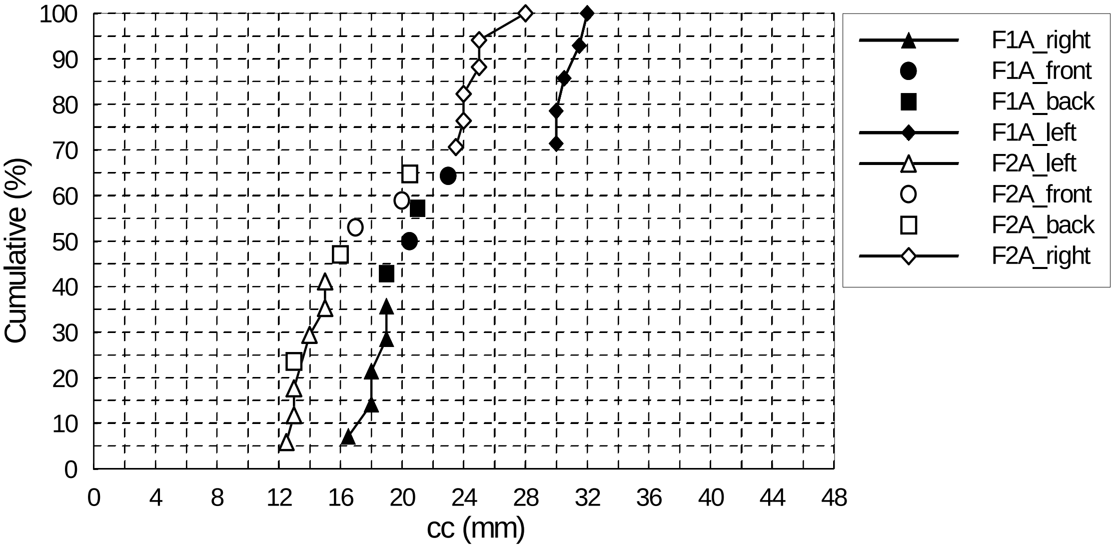
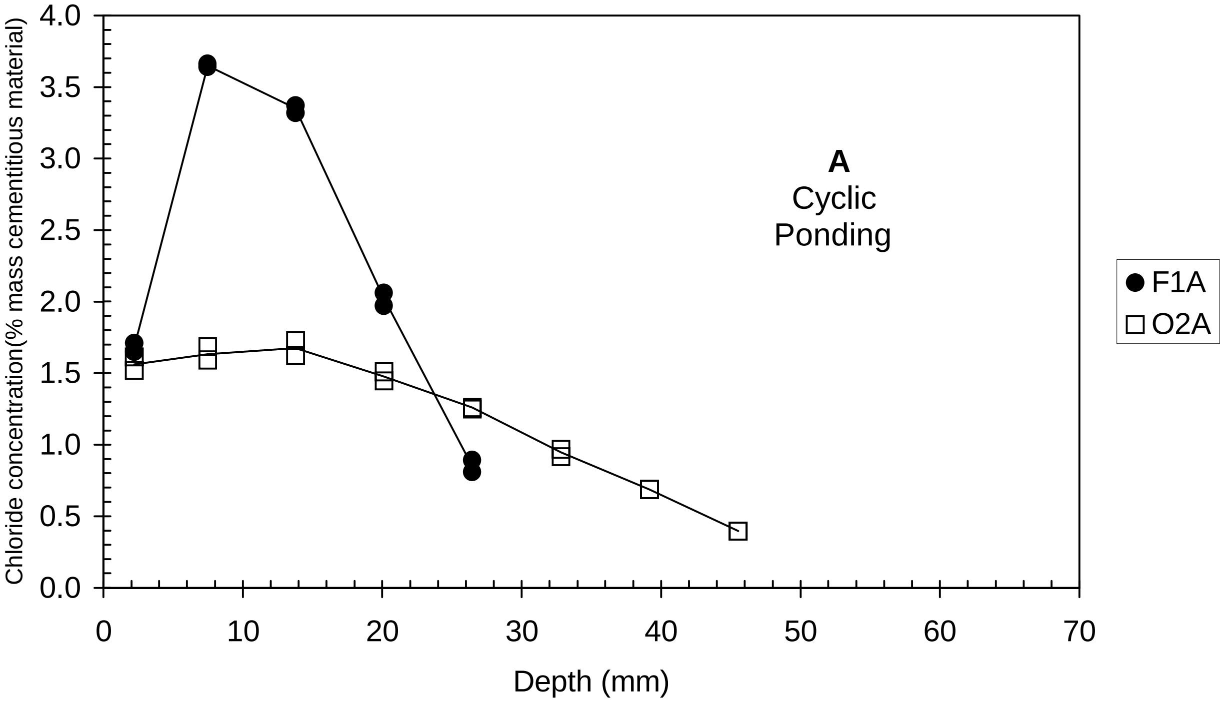


Figure 5. Distribution of concrete cover for specimens F1A and F2A, using different symbols for spots corresponding to different cut edges of the specimens, according to the convention given in Figure 1.



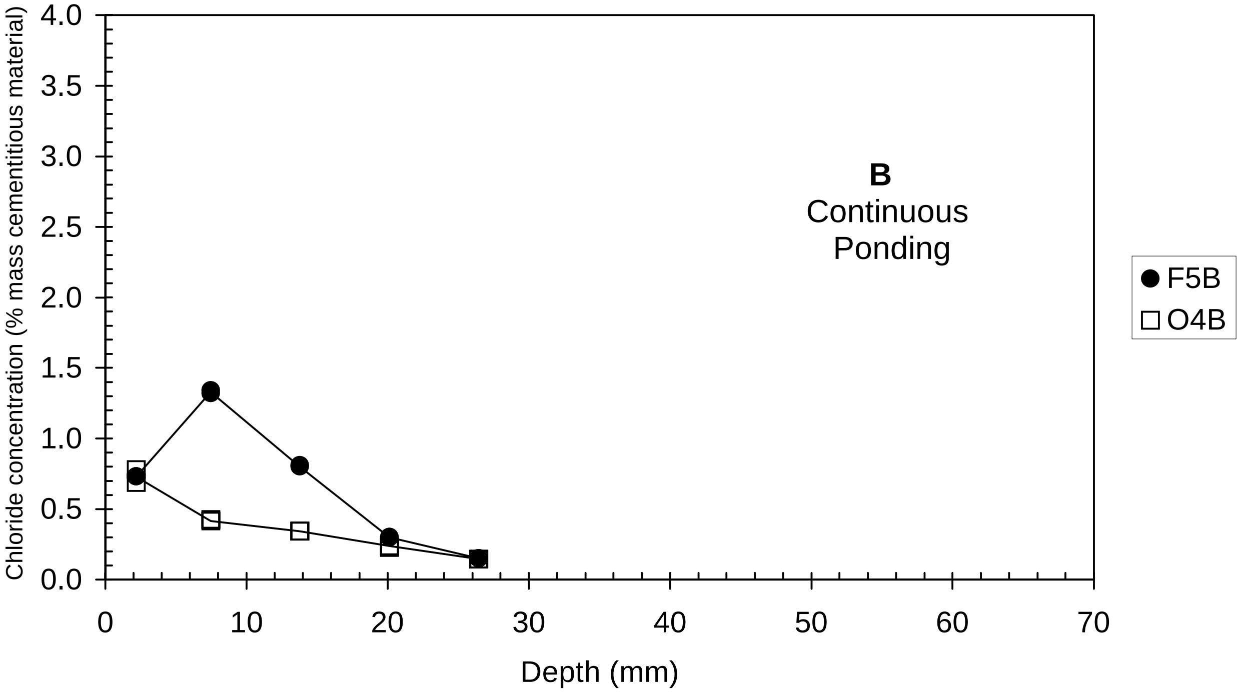


Figure 6. Chloride profiles for selected specimens of set A (756 days of exposure) and set B (390 days of exposure). Symbols show values of duplicate chloride analyses of each concrete powder sample.

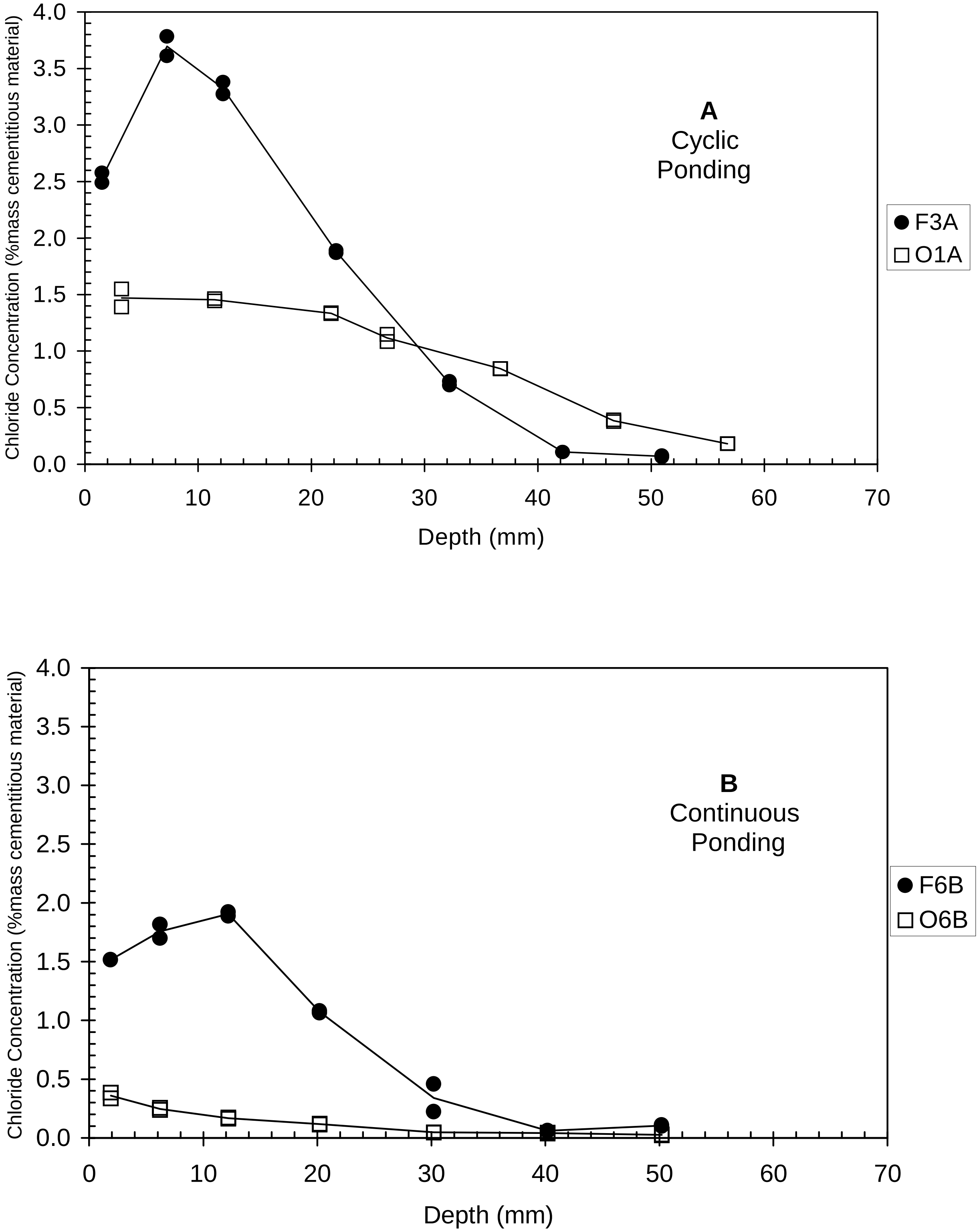


Figure 7. Chloride profiles for selected specimens of set A (1650 days of exposure) and set B (1200 days of exposure). Symbols show values of duplicate chloride analyses of each concrete powder sample.

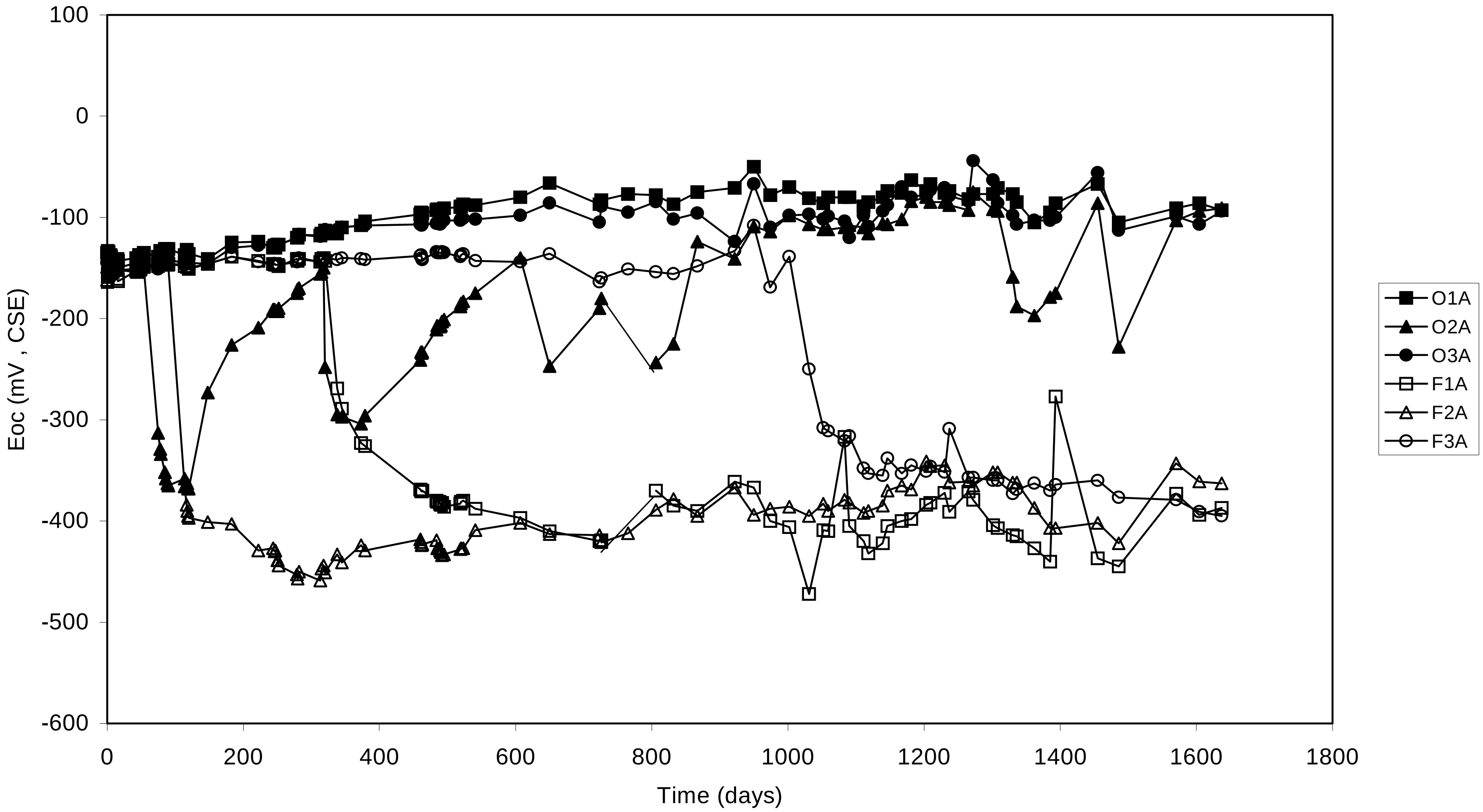


Figure 8. a. Open circuit potentials as a function of exposure time for specimens of set A (near the end of the wet cycle).

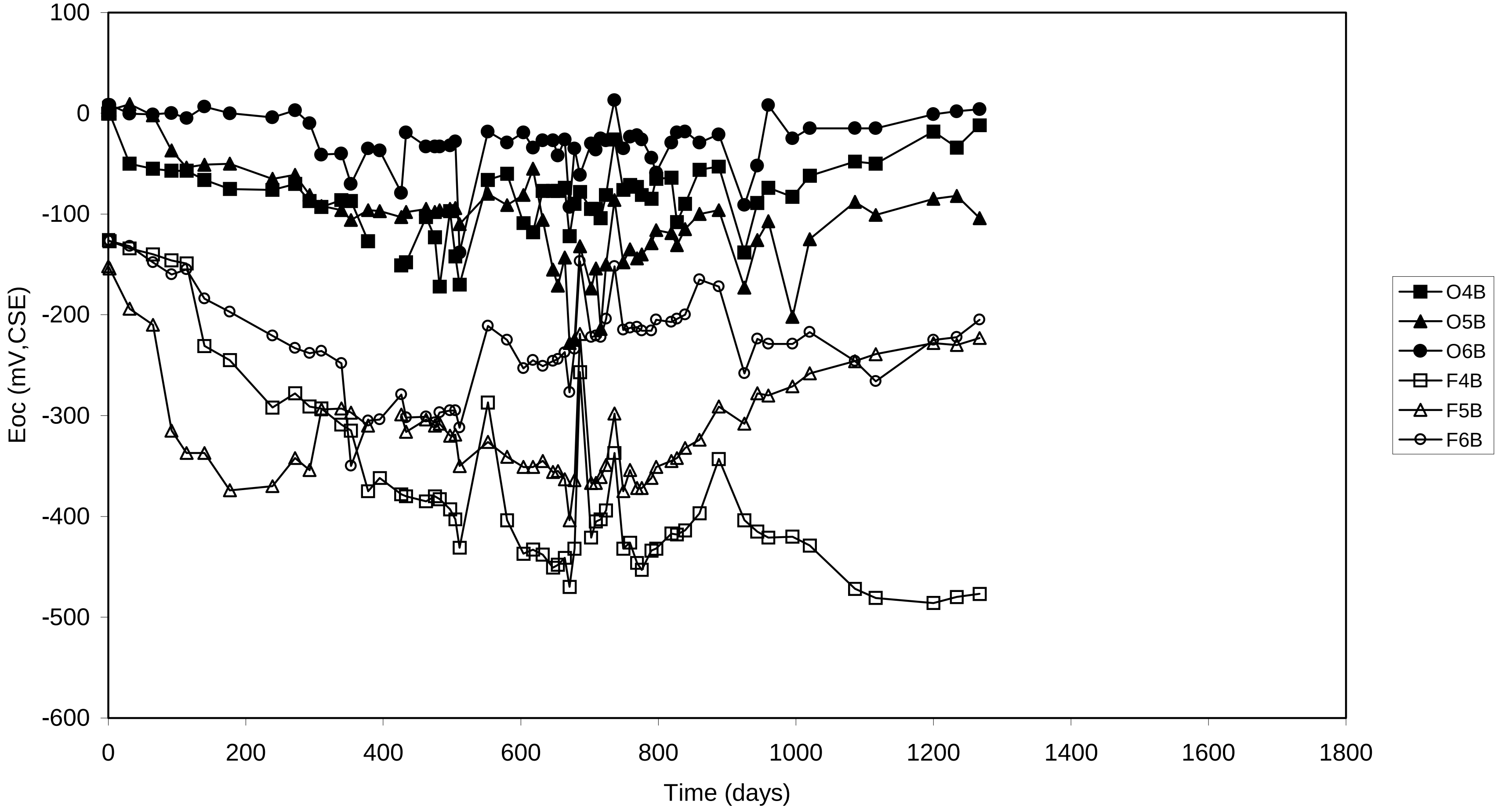
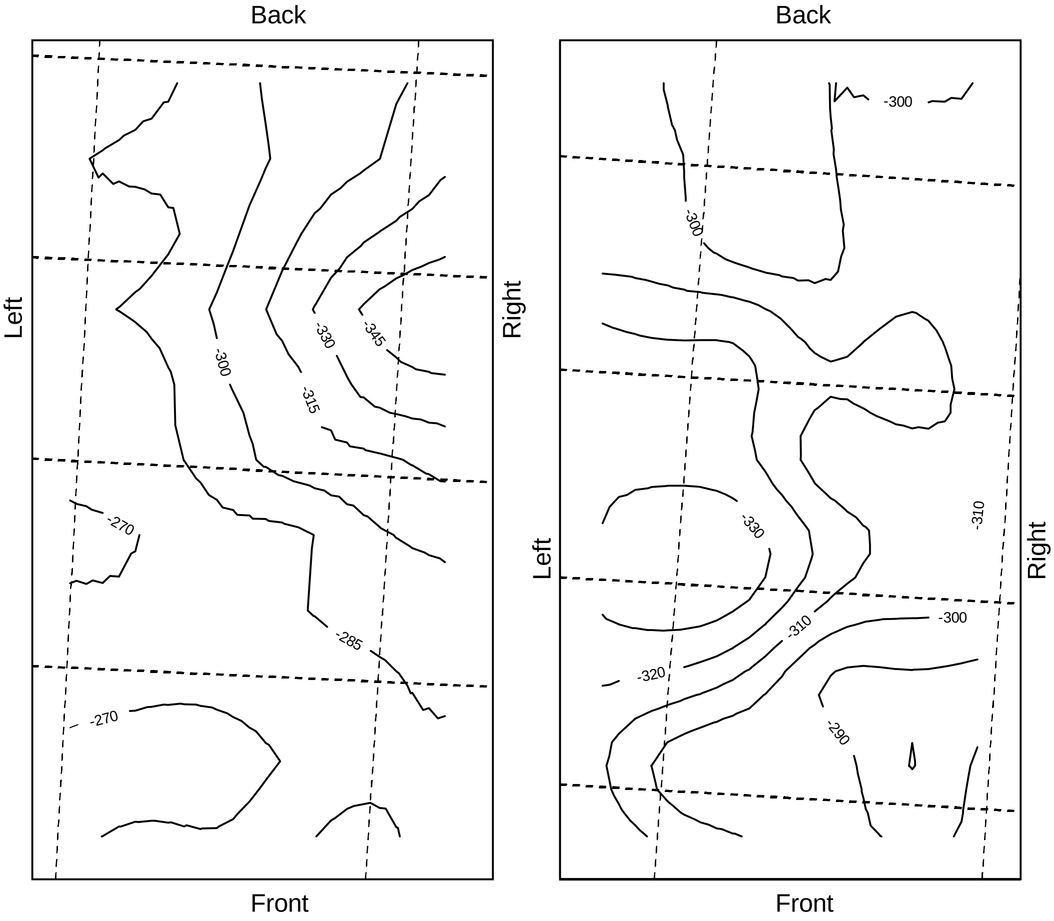


Figure 8. b. Open circuit potentials as a function of exposure time for specimens of set B.



(a)

(b)

Figure 9. Potential mapping obtained after 9 days in the 21st. dry cycle for specimens :
a) F1A and b) F2A . The equipotential lines are drawn inside a scaled plan view of the dam footprint. The labels (front, back, left and right) in the border of each map are given to define the orientation of the specimens, according to Figure 1. Potentials vs. SCE.

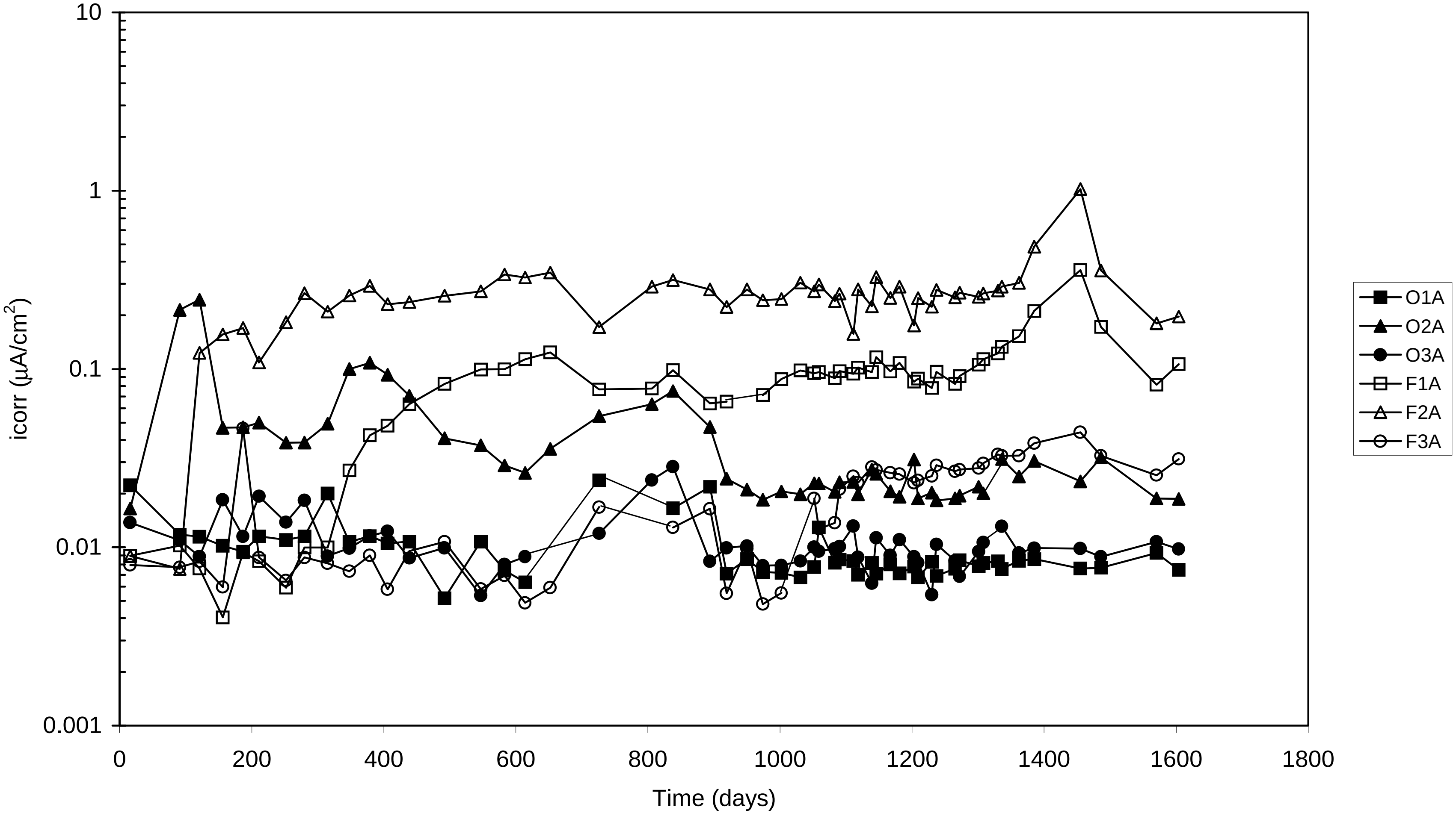


Figure 10. a. Evolution of nominal corrosion current density as a function of exposure time for specimens of set A (near the end of the wet cycle).

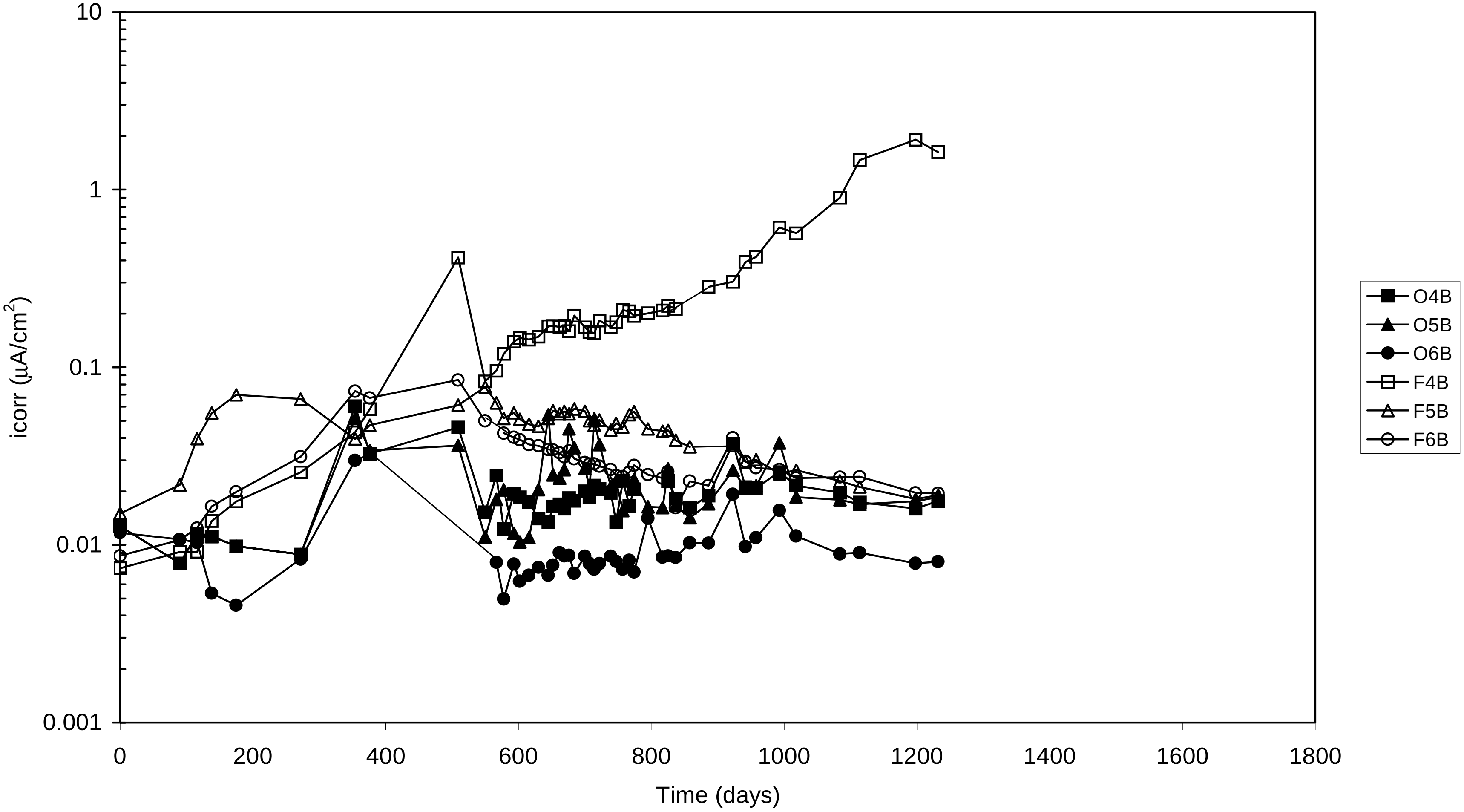


Figure 10. b. Evolution of nominal corrosion current density as a function of exposure time for specimens of set B.

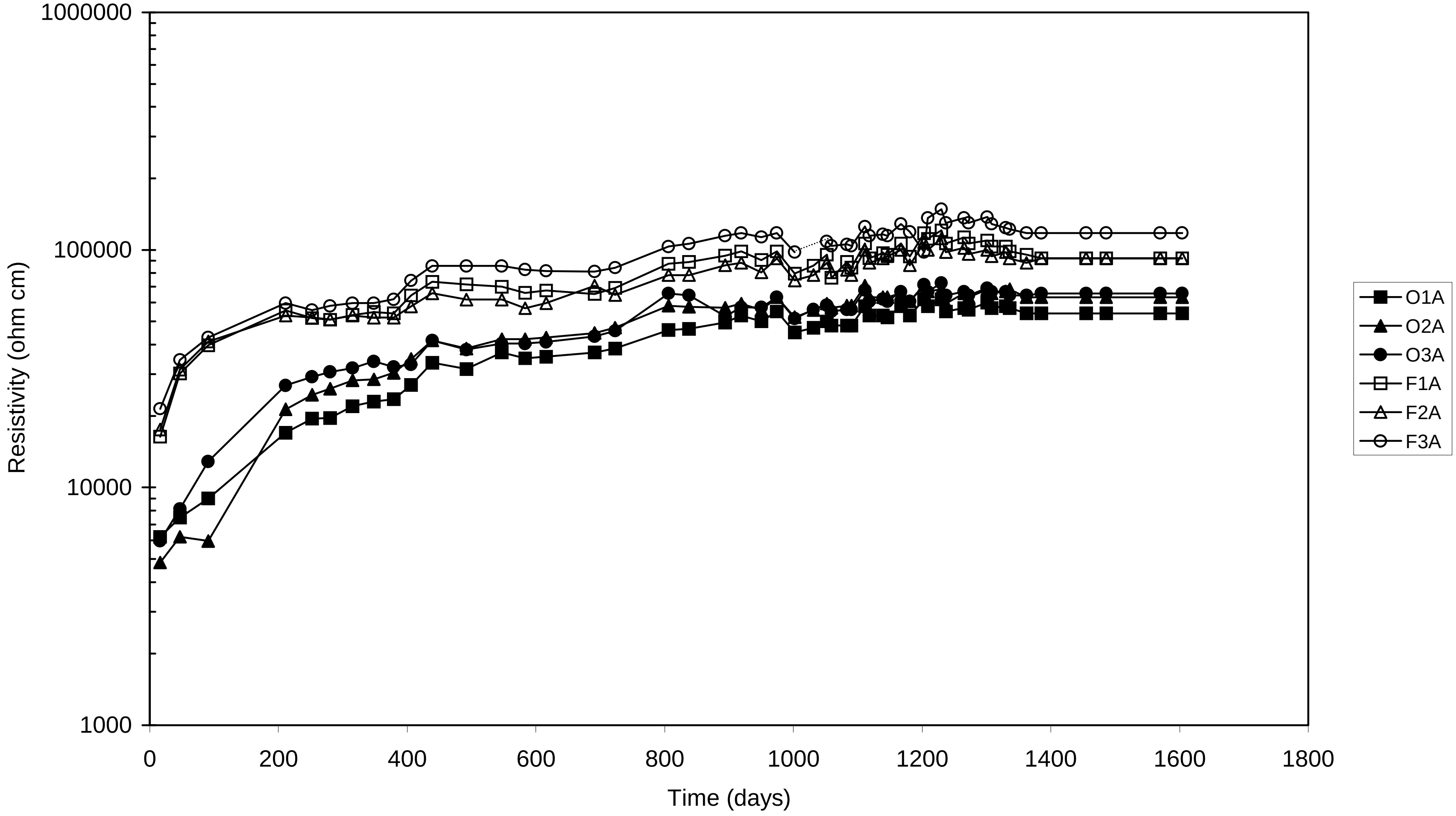


Figure 11. a. Evolution of concrete resistivity as a function of time for specimens of set A (during the wet cycles).

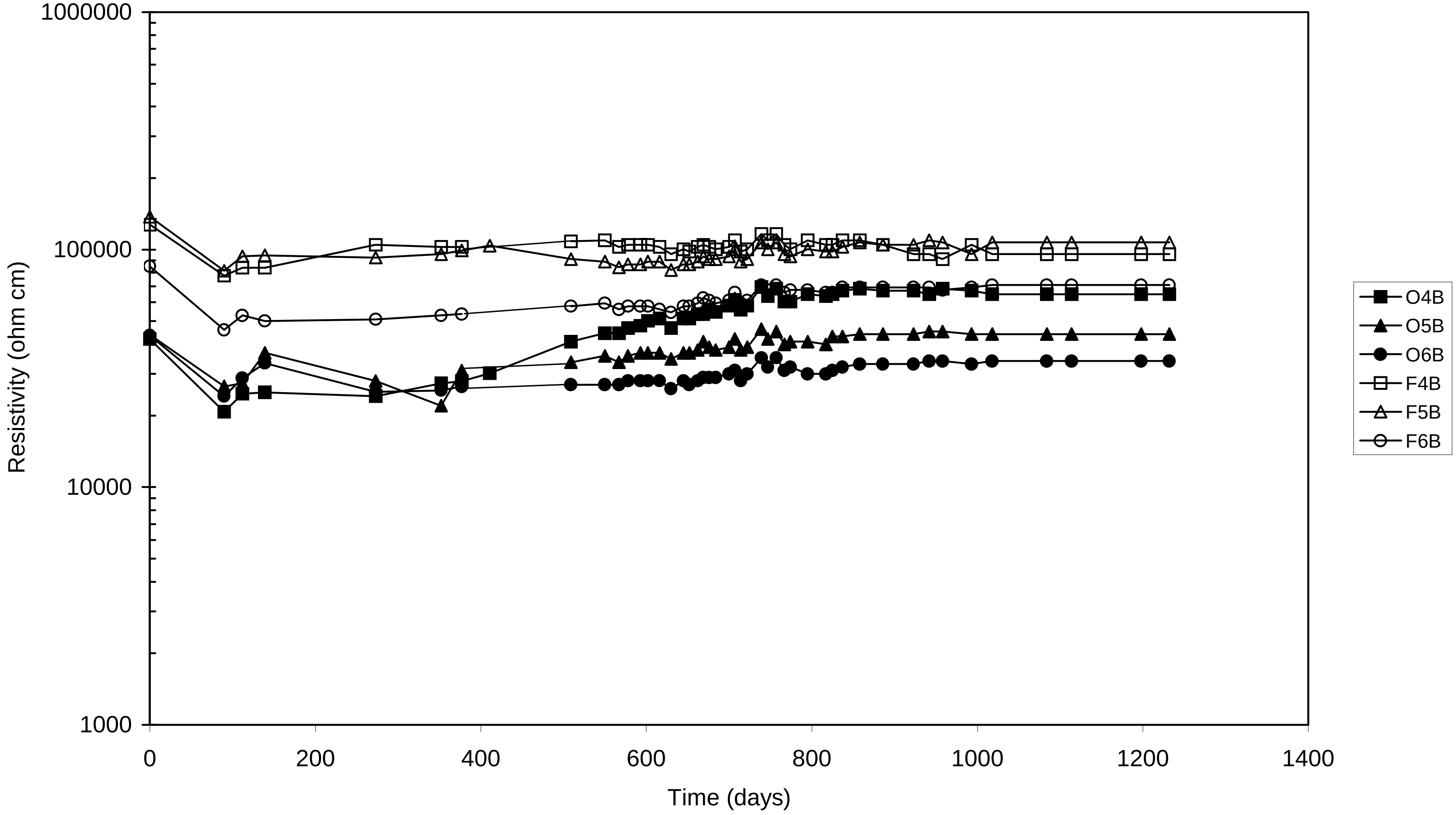


Figure 11. b. Evolution of concrete resisitivity as a function of time for specimens of set B (permanently wet, with 8 months fresh water preconditioning before day 0).

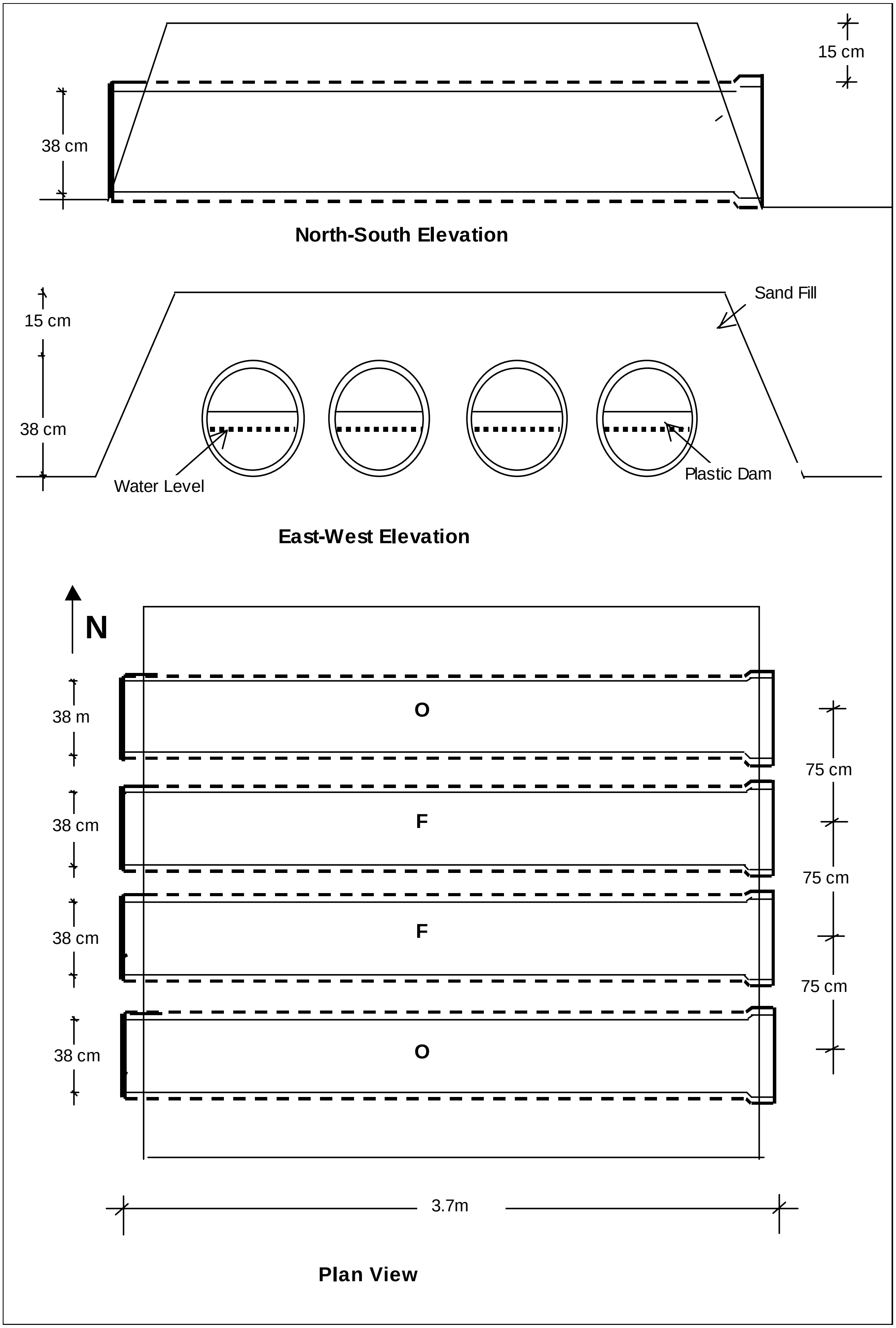


Figure 12. Culvert Yard Test Set up.

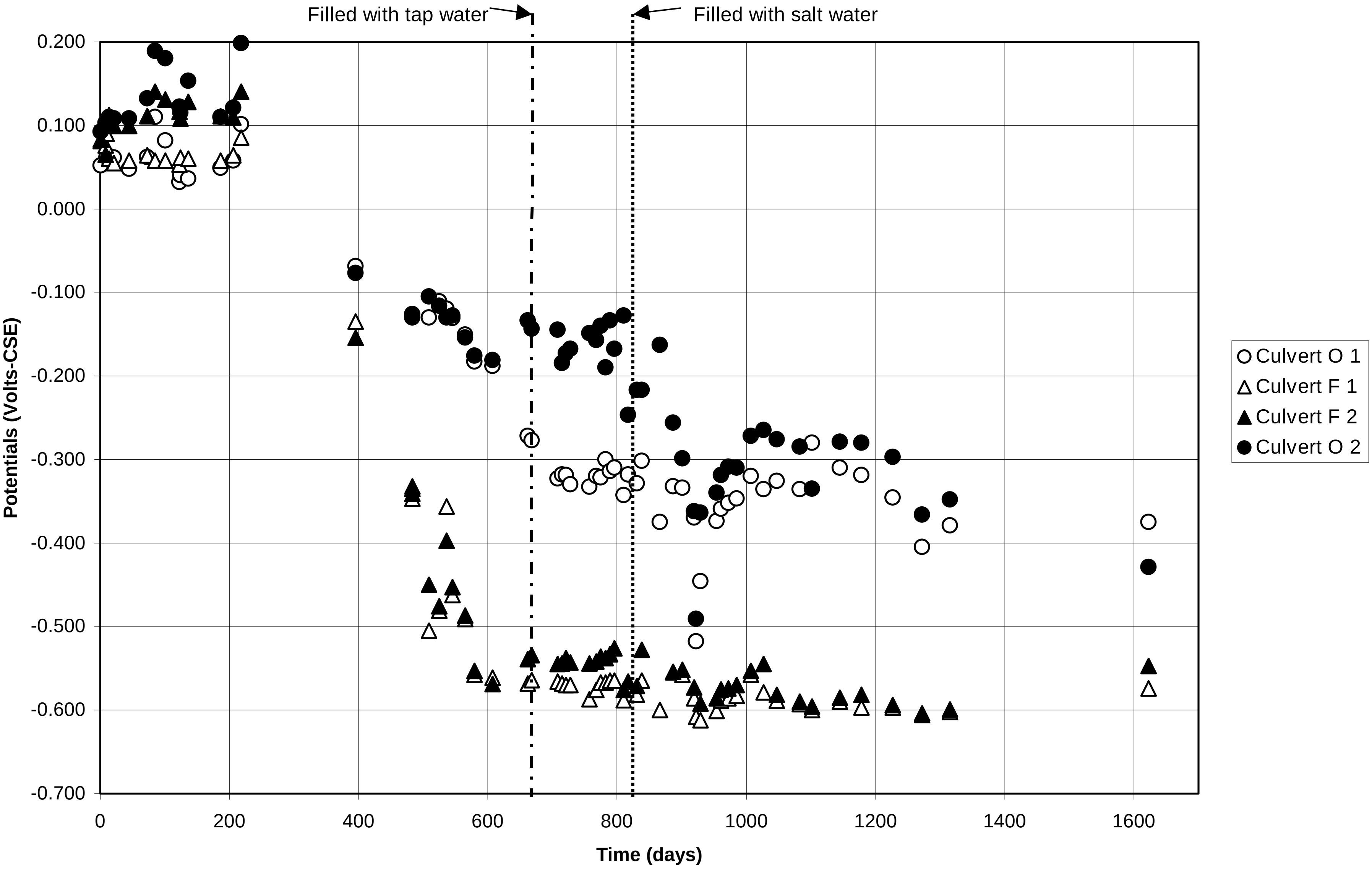


Figure 13. Potential as a function of time for reinforcement in each pipe.

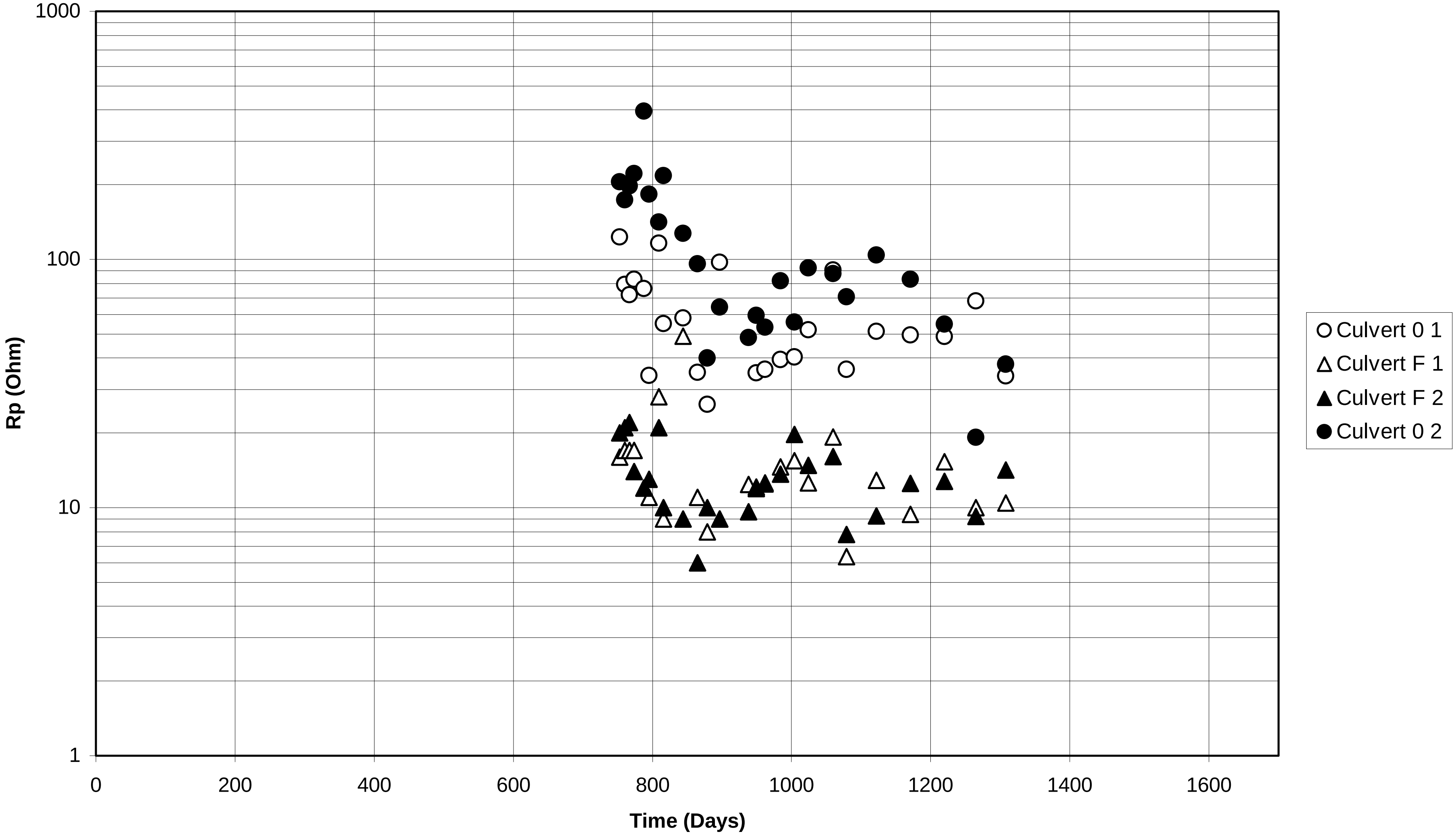


Figure 14 . Polarization resistance as a function of time for each pipe (solution resistance (solution resistance subtracted).

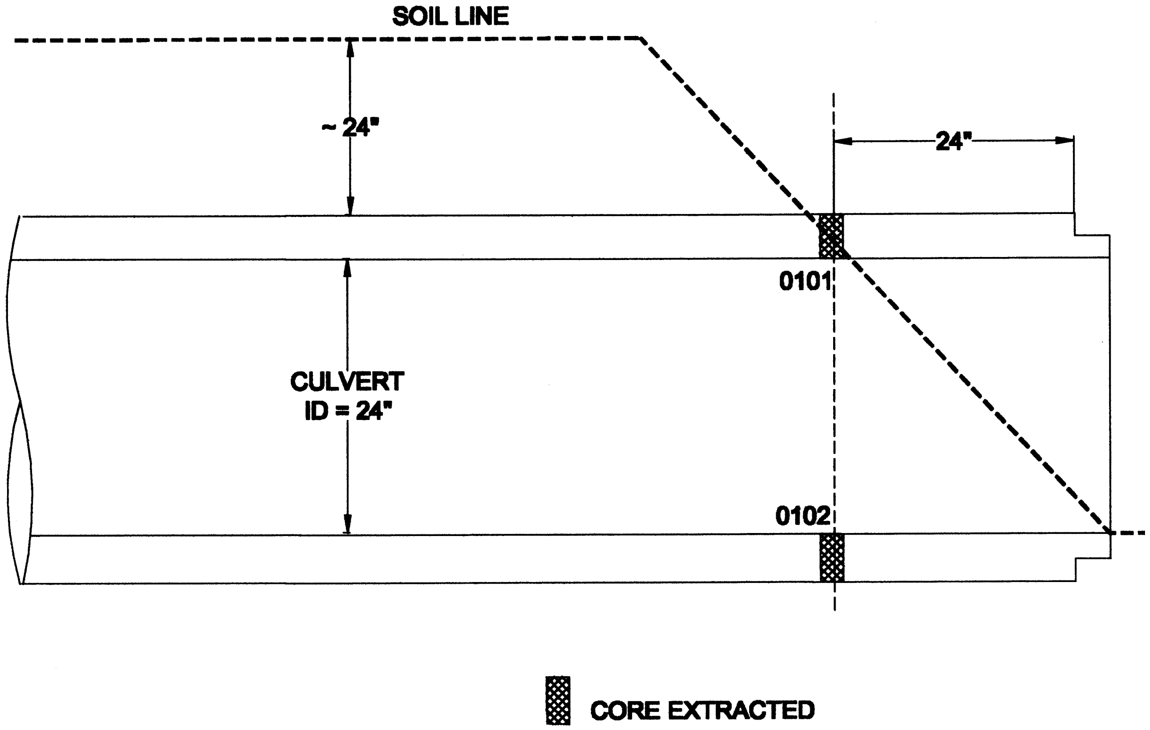


Figure 15. Site No. 01

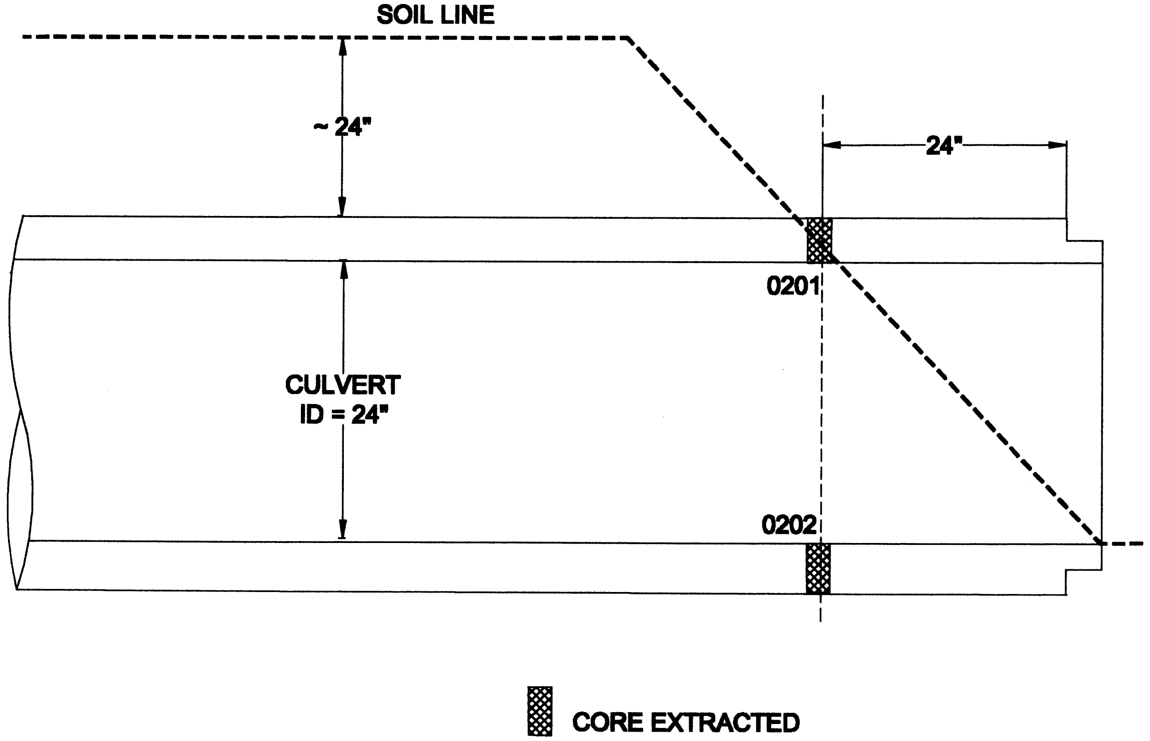


Figure 16. Site No. 02

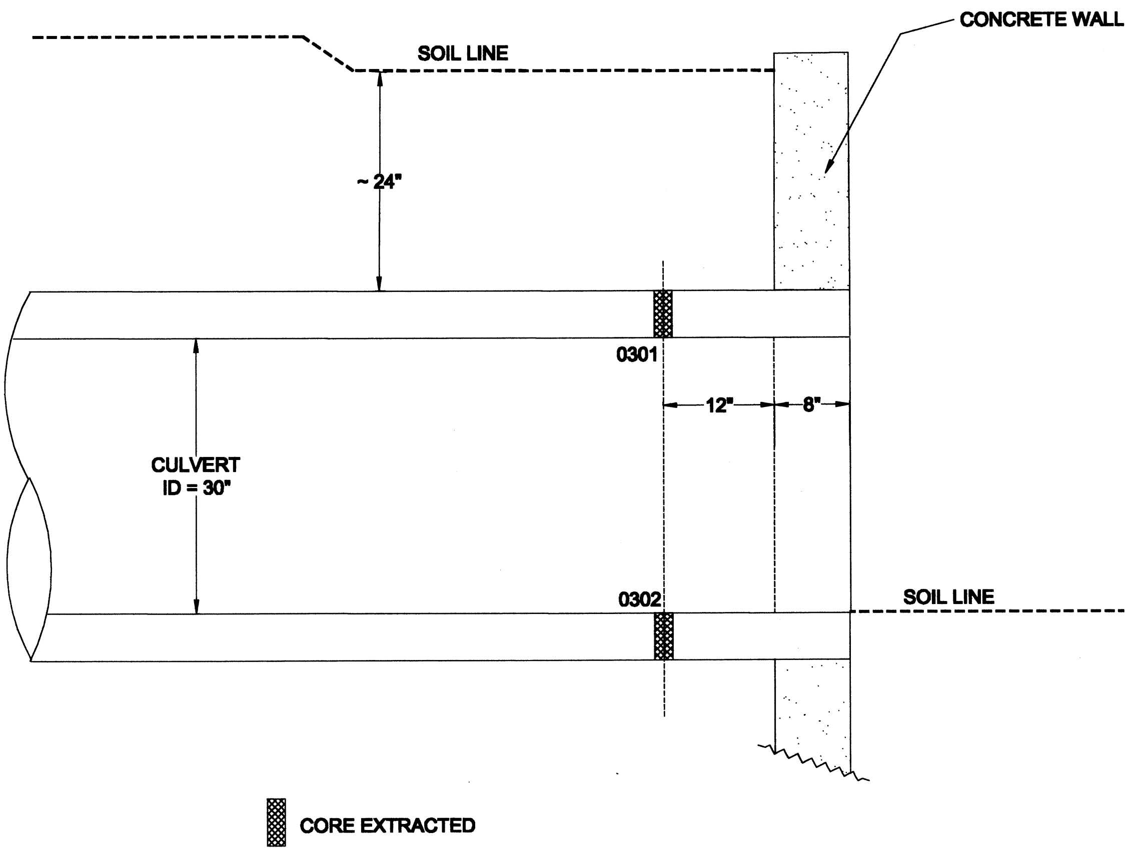


Figure 17 Site No. 03

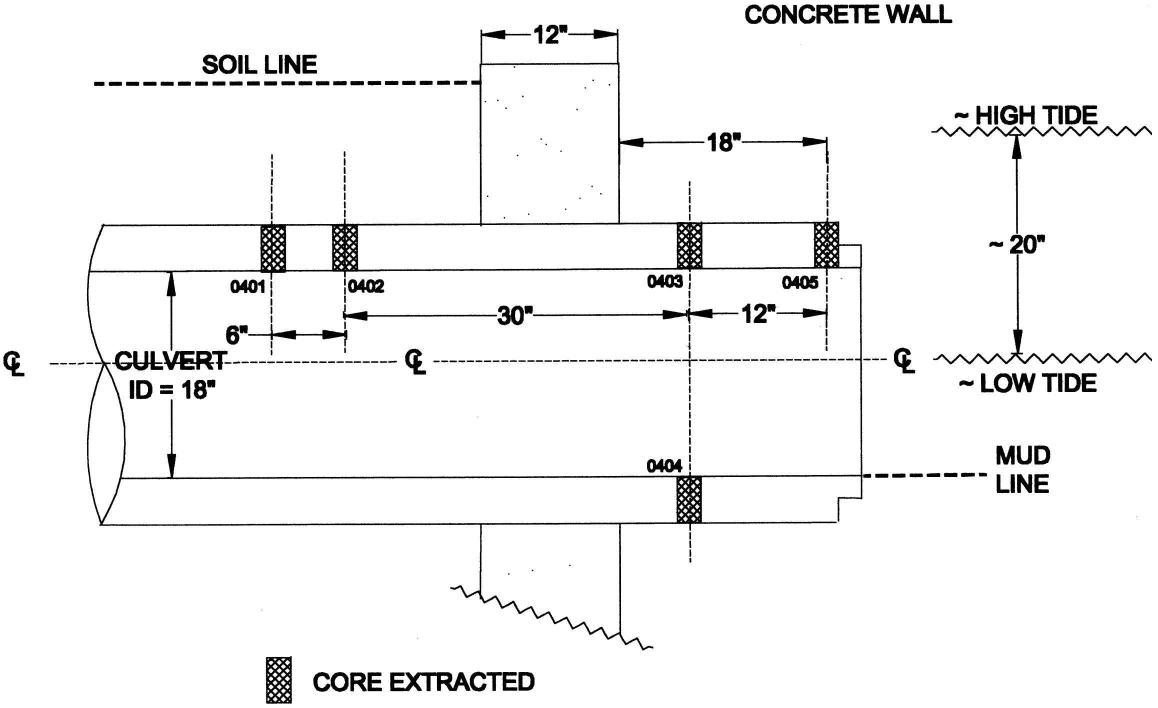


Figure 18 Site No. 04

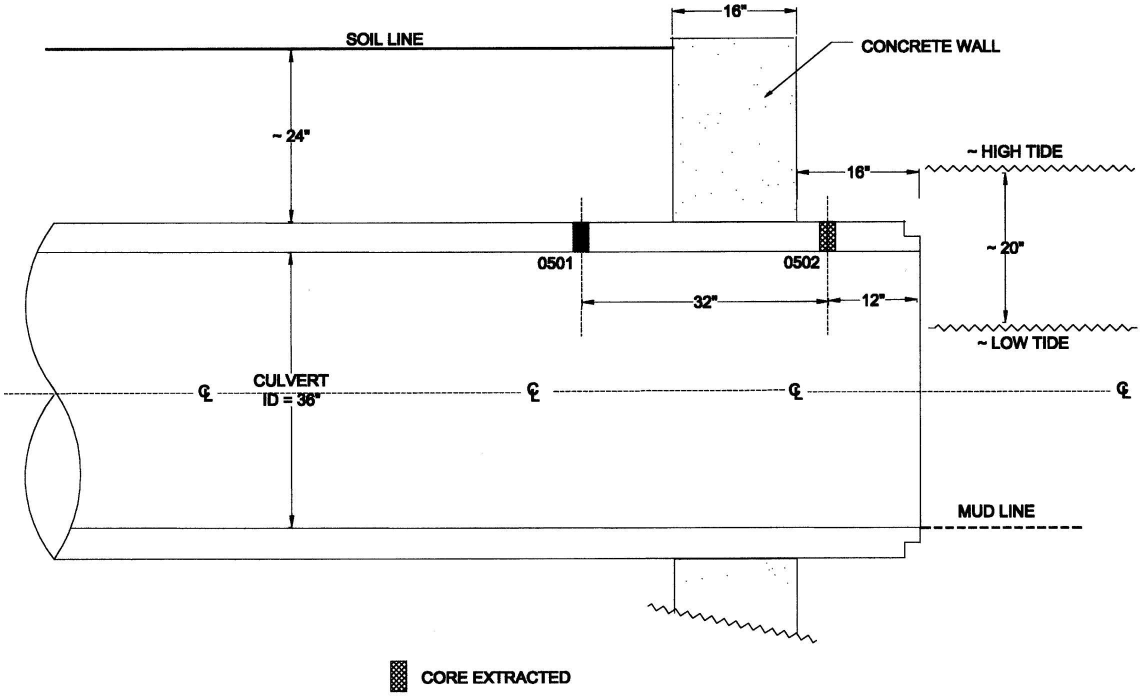


Figure 19 Site No. 05

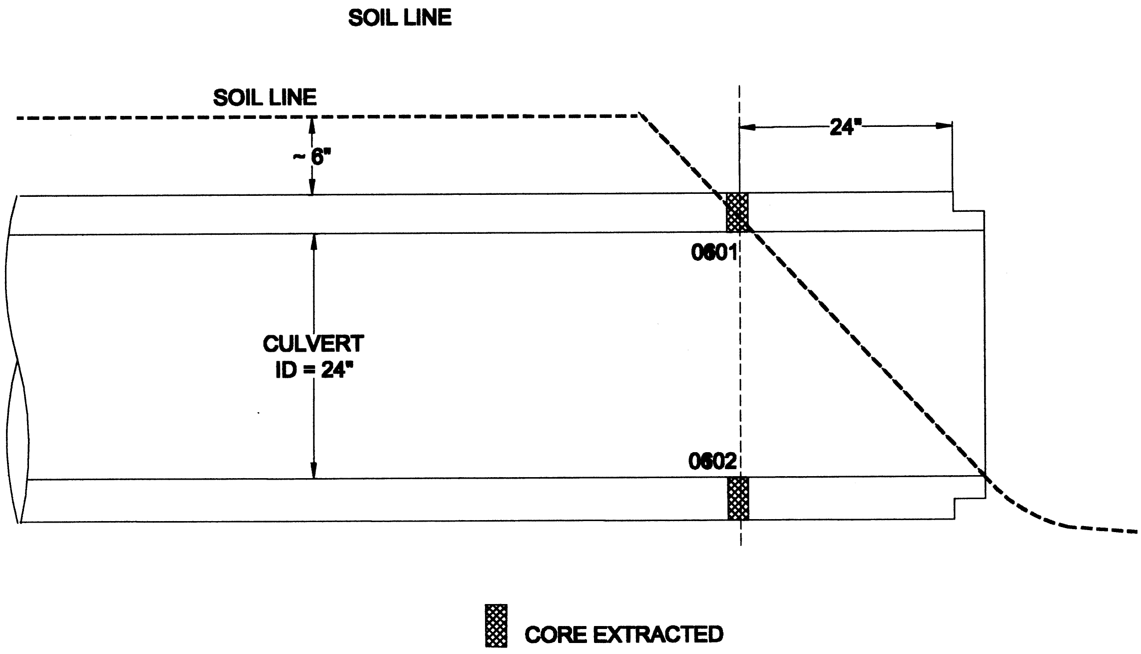


Figure 20. Site No. 06

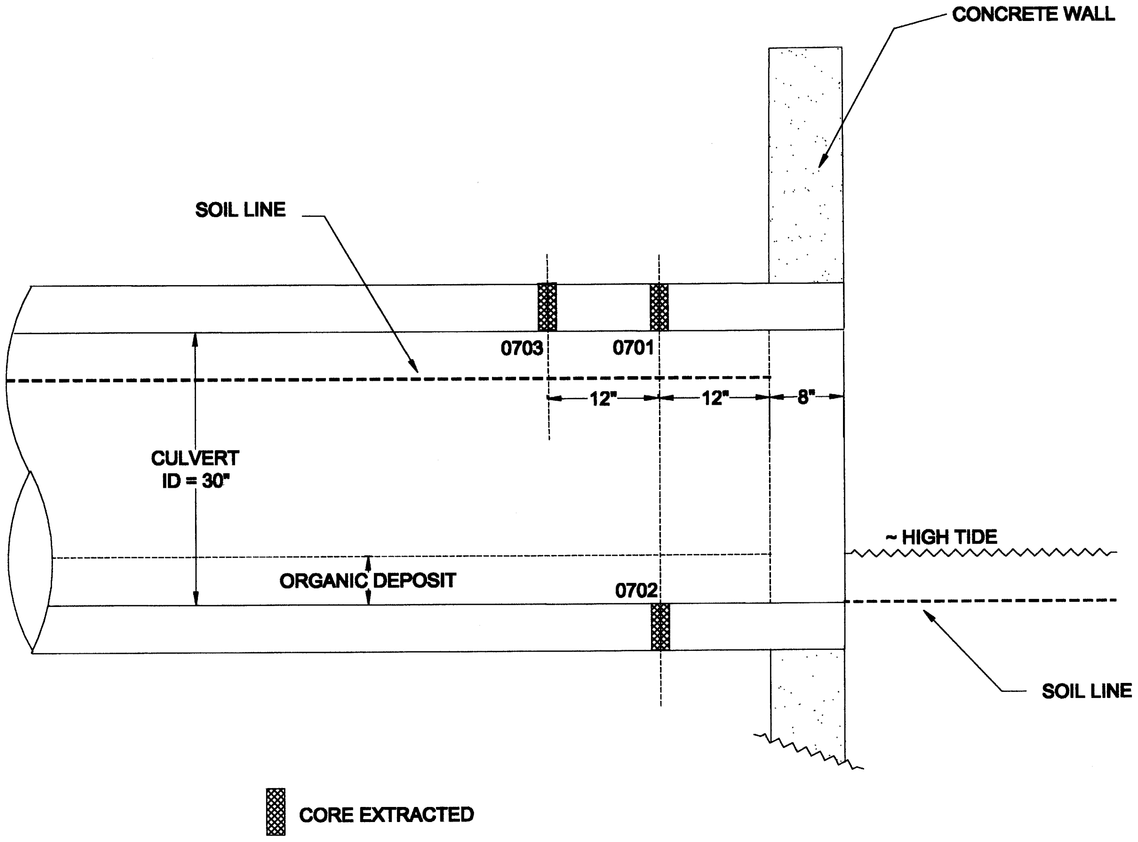


Figure 21. Site No. 07

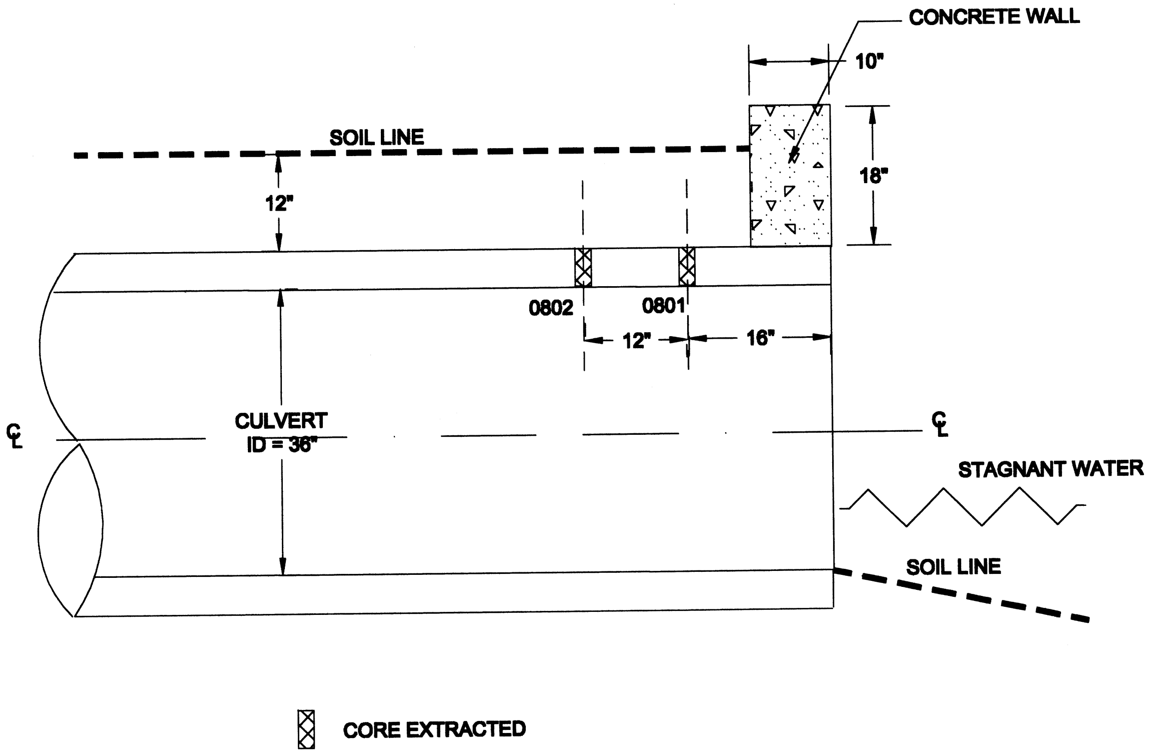
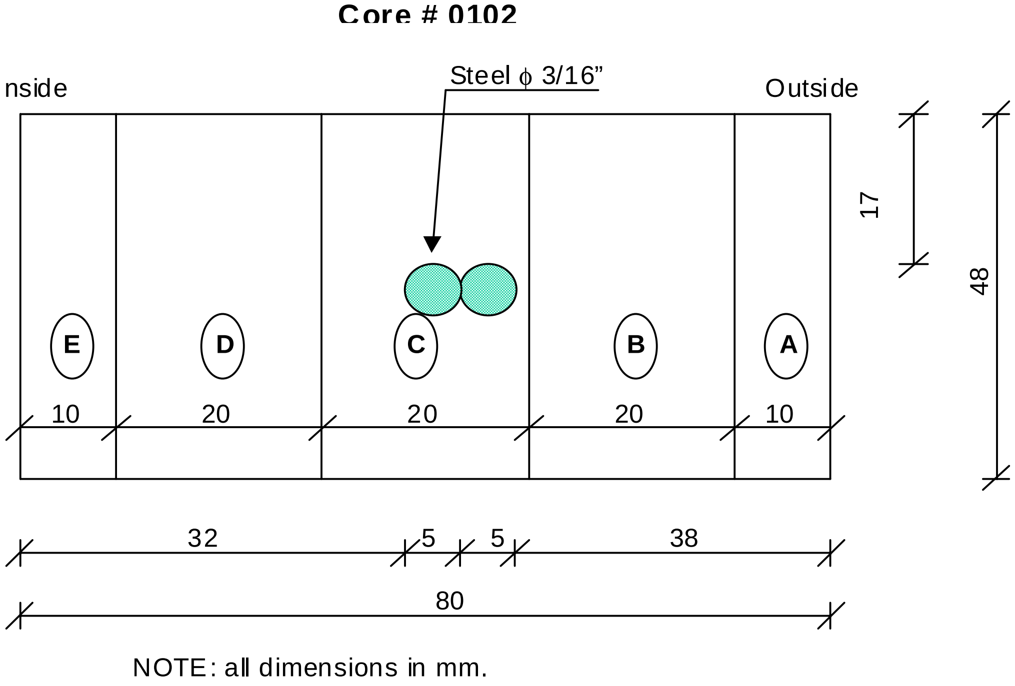


Figure 22. Site No. 08



Sample #	Sample Width (cm)	Distance x (cm)	Chloride Content (kg/m^3)
0102-E	1.0	0.4	0.14
0102-D	2.0	1.9	0.09
0102-C	2.0	3.9	0.05
0102-B	2.0	5.9	0.065
0102-A	1.0	7.4	0.16

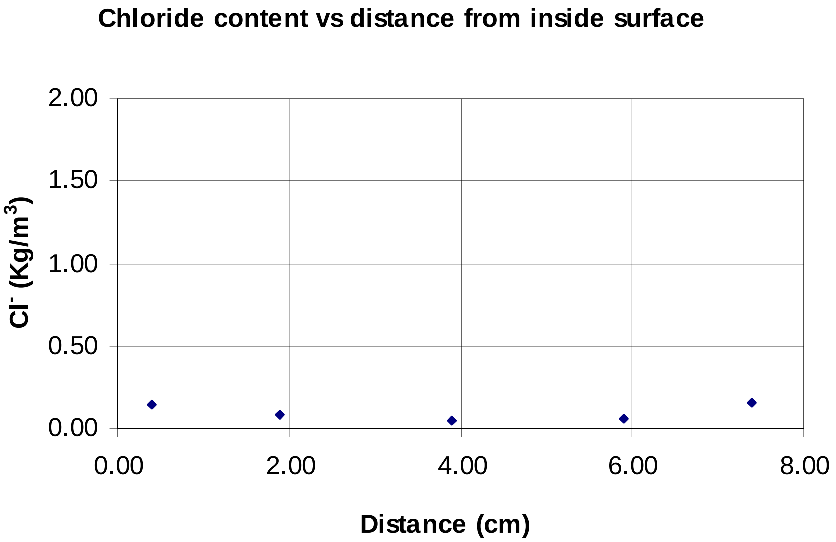
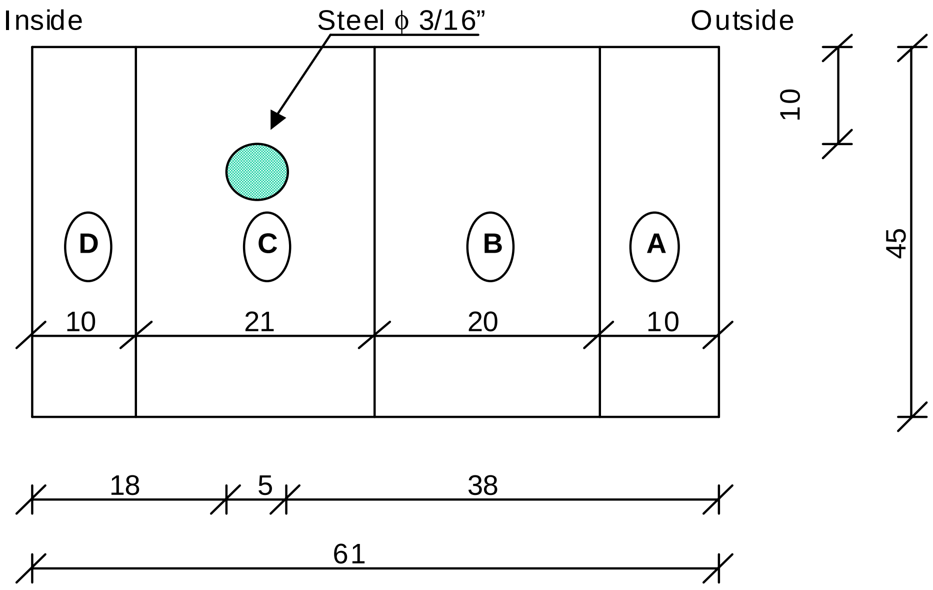


Figure 23. Core # 0102

Core # 0401



NOTE: all dimensions in mm.

Sample #	Sample Width (cm)	Distance x (cm)	Chloride Content (kg/m^3)
0401-D	1.0	0.4	17.71
0401-C	2.0	1.9	14.84
0401-B	2.0	4.0	9.26
0401-A	1.0	5.5	7.44

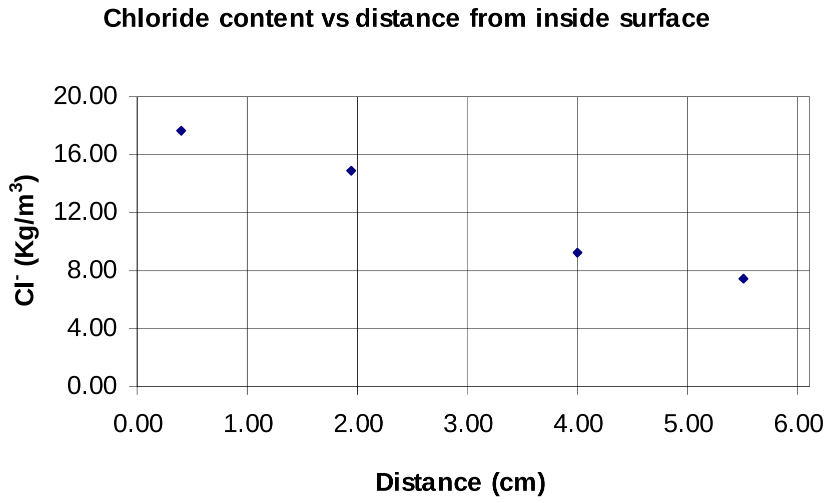
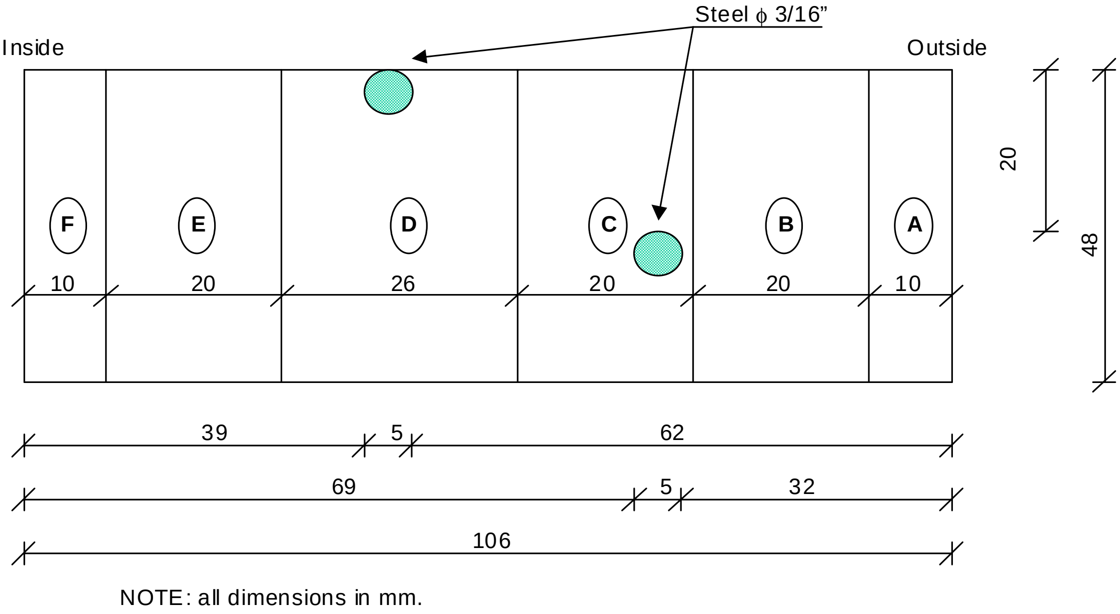


Figure 24. Core # 0401



Sample #	Sample Width (cm)	Distance x (cm)	Chloride Content (kg/m^3)
0502-F	1.0	0.4	12.1
0502-E	2.0	1.9	11.71
0502-D	2.6	4.2	10.64
0502-C	2.0	6.5	8.65
0502-B	2.0	8.5	10.76
0502-A	1.0	10.0	11.36

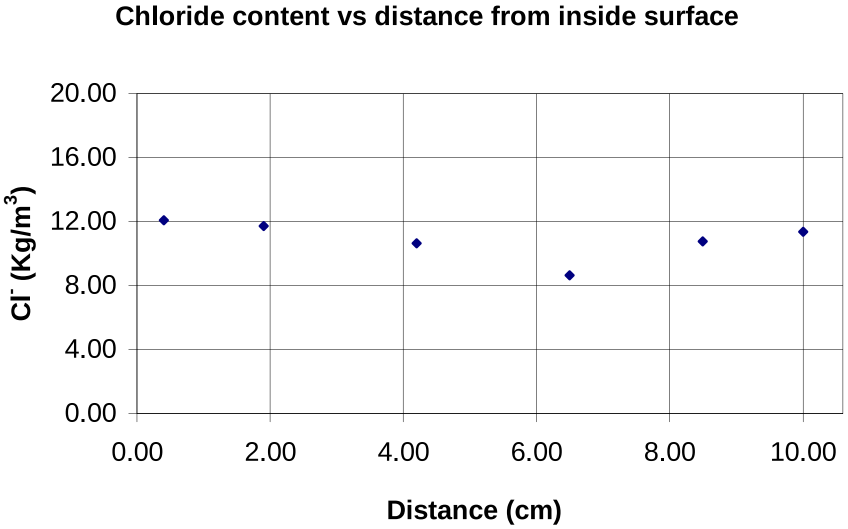
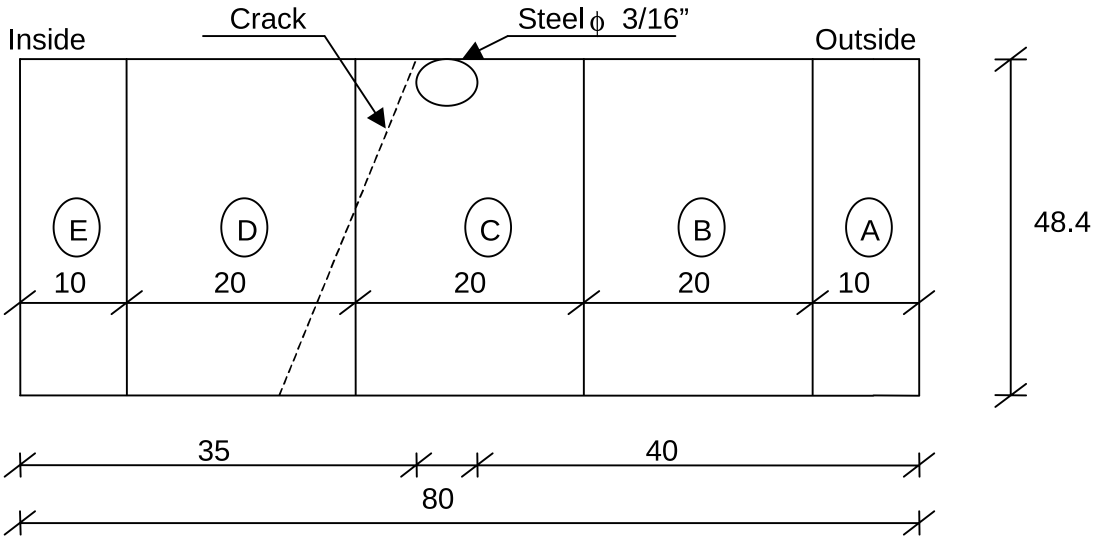


Figure 25. Core # 0502



NOTE: all dimensions in mm.

Sample #	Distance x (cm)	Chloride Content (kg/m^3)
0602-E	0.5	11.09
0602-D	2.0	11.05
0602-C	4.0	11.55
0602-B	6.0	10.07
0602-A	7.5	11.96

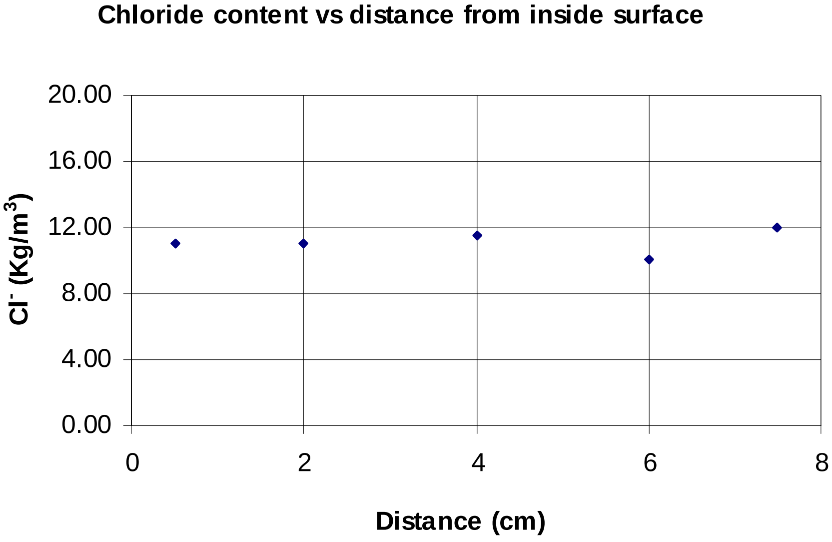
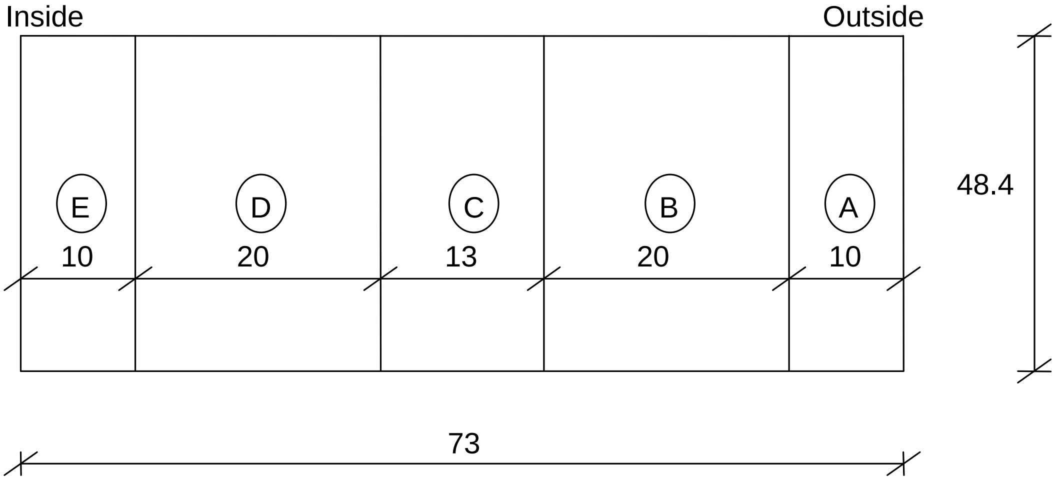


Figure 26. Core # 0602



NOTE: all dimensions in mm.

Sample #	Distance x (cm)	Chloride Content (kg/m^3)
0702-E	0.5	14.43
0702-D	2.0	13.31
0702-C	3.6	12.30
0702-B	5.3	12.76
0702-A	6.8	16.83

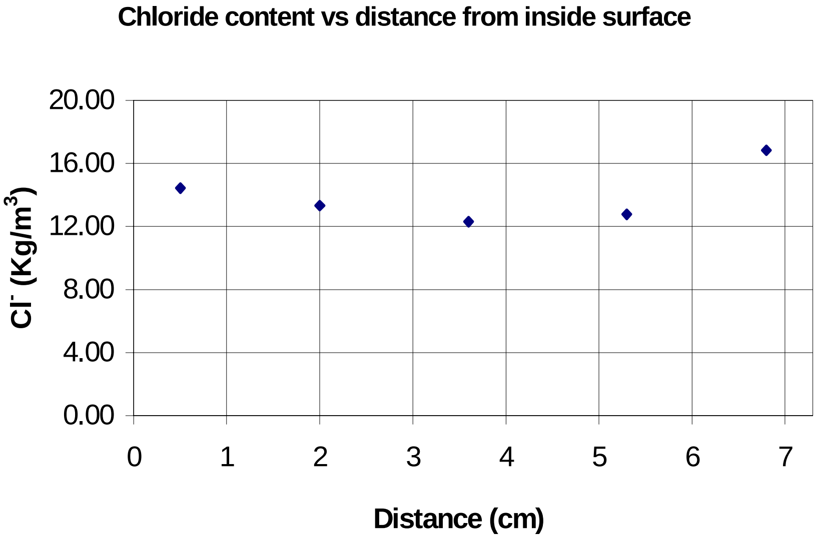
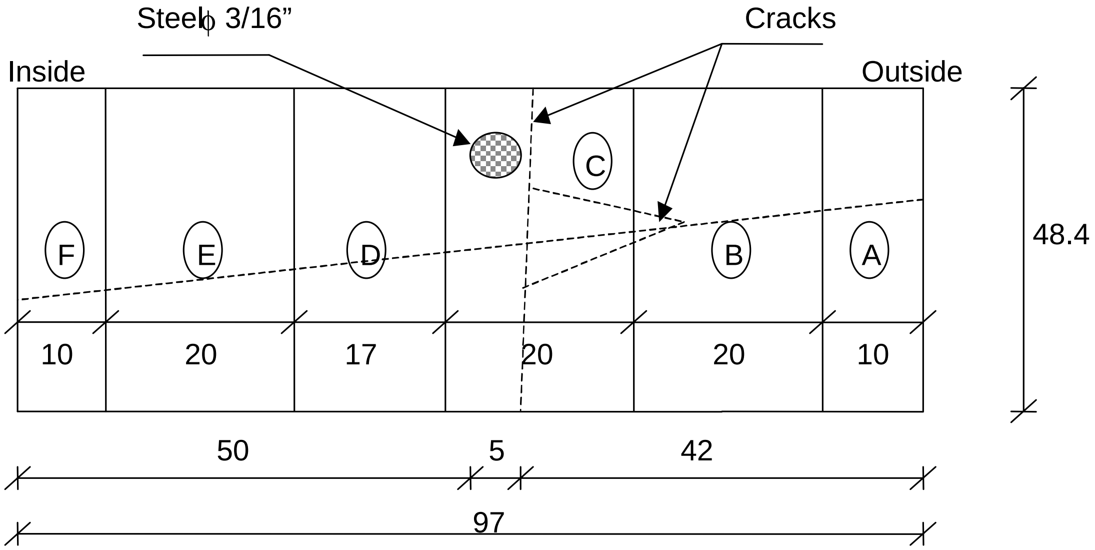


Figure 27. Core # 0702



NOTE: all dimensions in mm

Sample #	Distance x (cm)	Chloride Content (kg/m^3)
0703-F	0.40	13.85
0703-E	2.0	13.38
0703-D	3.8	9.99
0703-C	5.7	8.39
0703-B	7.7	9.17
0703-A	9.2	10.53

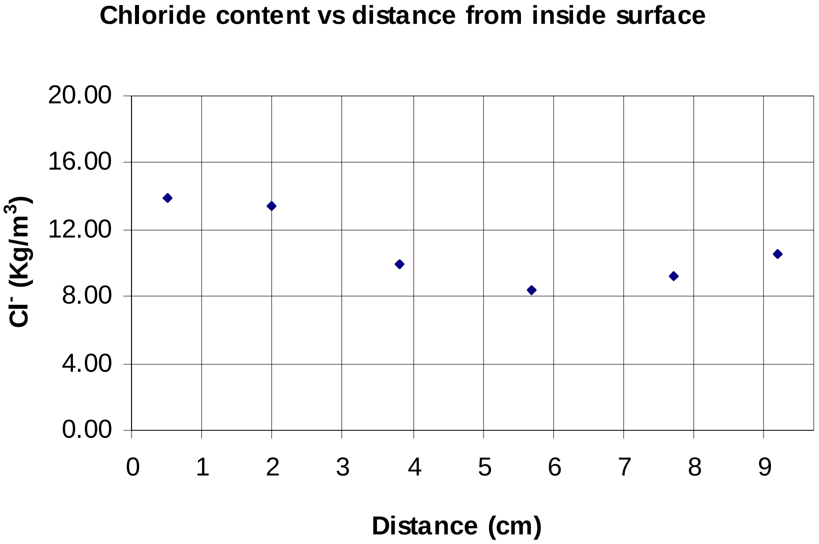
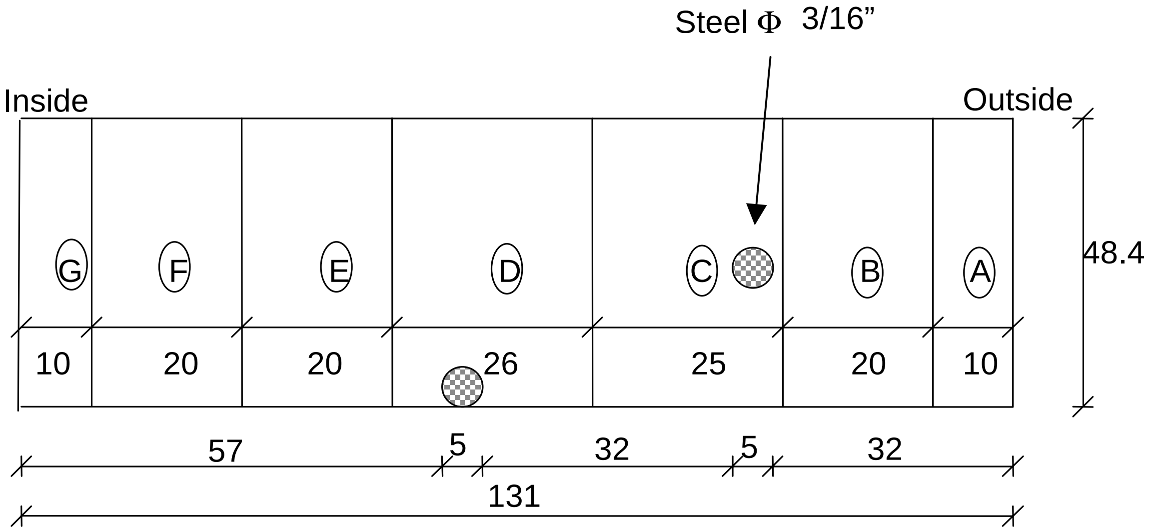


Figure 28. Core 0703



NOTE: all dimensions in mm.

Sample #	Distance x (cm)	Chloride Content (kg/m^3)
0801-G	0.40	0.2
0801-F	1.90	0.17
0801-E	3.90	0.015
0801-D	6.20	0.018
0801-C	8.75	0.07
0801-B	11.00	0.00
0801-A	11.50	0.14

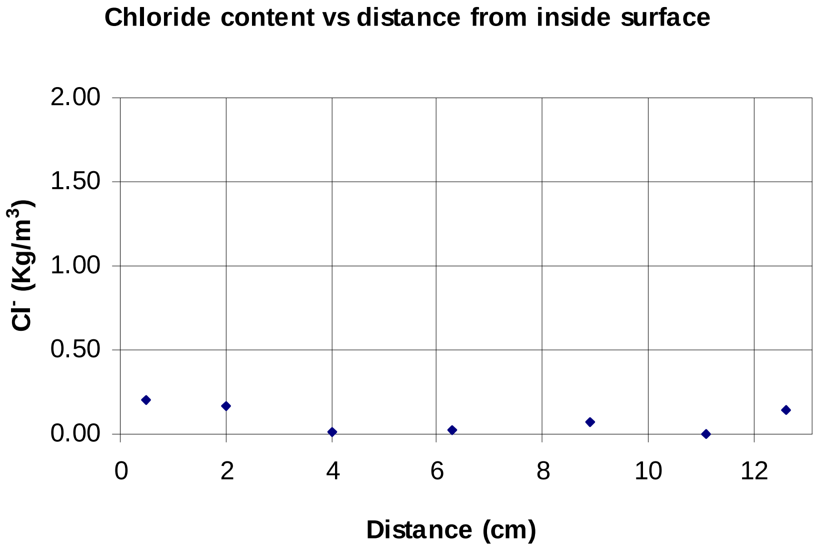


Figure 29. Core # 0801

Figure 30. Site No. 7. End of San Jose Street. Low Tide. Core extraction. Evidence of pipe distress.

Figure 31. Site No. 7. End of San Jose Street. Low Tide. Side View.

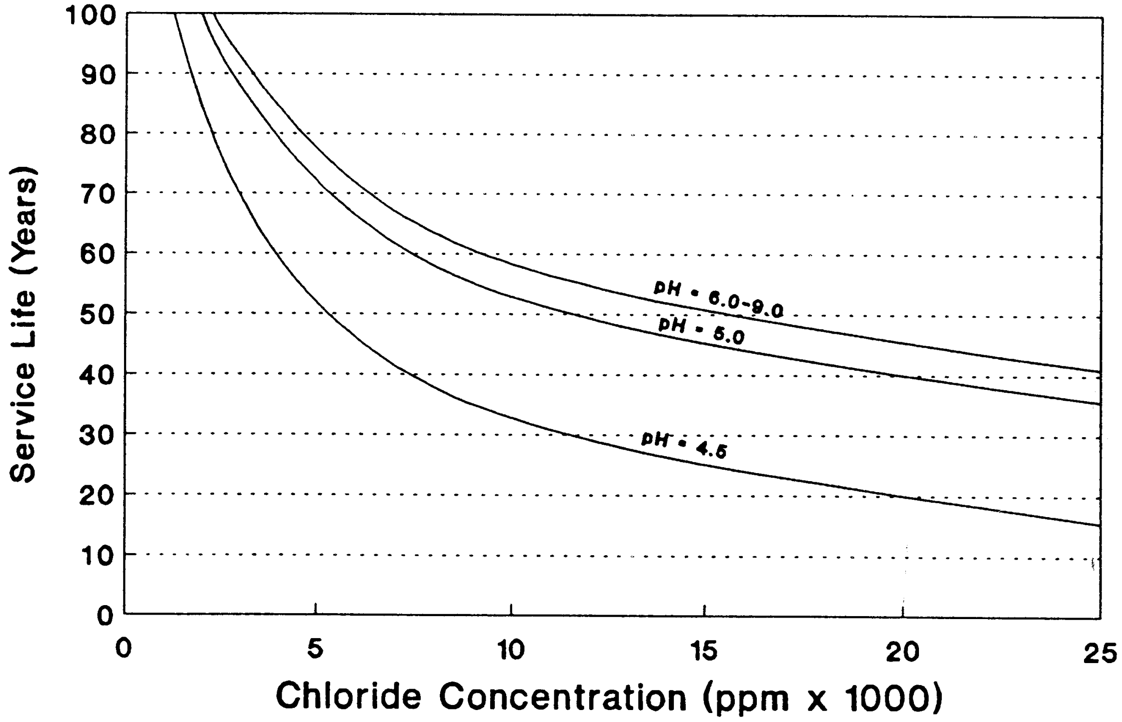


Figure 32. Estimated Service Life vs. pH and Chlorides for 36" DIA Concrete Culverts, S=1500 ppm

CONVERSION FACTORS, US CUSTOMARY TO METRIC UNITS

Multiply	***by***	***to obtain***
inch	25.4	mm
foot	0.3048	meter
square inches	645	square mm
cubic yard	0.765	cubic meter
pound/cubic yard	0.593	kg/cubic meter
gallon/cubic yard	4.95	liter/cubic meter
standard cubic feet/hour	466.67	ml/minute
ounces	28.35	gram
pound	0.454	kilogram
pound (lb)	4.448	newtons
kip (1000 lb)	4.448	kilo Newton (kN)
pound/in^2	0.0069	MPa
kip/in^2	6.895	MPa
ft-kip	1.356	kN-m
in-kip	0.113	kN-m