Permeability Evaluation of Concrete Bridge Structures Exposed to Marine Environment in Florida

CONSTANTINE A. MELETIOU AND MANG TIA

A field permeability test (FPT) apparatus and method were developed and evaluated in both the laboratory and the field. The developed prototype FPT apparatus and method were used in the testing and evaluation of in-service marine structures in conjunction with other standard tests. The developed FPT apparatus and method appear promising in providing a suitable measuring system for the rapid, convenient, and reliable determination of the in situ water permeability of structural concrete. There appears to be a linear relationship between the charge (in coulombs) passed through a concrete material as measured by the rapid chloride permeability test (AASHTO T277-83) and its corresponding water permeability, with a coefficient of determination, R^2, of 0.90. There was an apparent relationship between permeability and durability of concrete in service. The concrete material that exhibited durability problems also demonstrated high permeability. The FPT method was demonstrated to be able to provide a relative measurement of permeability that can be used as an indication of the quality and performance characteristics of structural concrete.

Long-term durability of concrete structures in a marine environment has been a great concern to civil engineers in recent years as an increasing number of cases of premature deterioration of such structures have been reported throughout the country and the world. To ensure long-term durability of concrete structures, attention must be given to the quality of the concrete material with regard to those factors that may affect its performance under the given exposure conditions.

Concrete exposed to a marine environment may deteriorate as a result of the combined action of chemical and physical forces. These include chemical action of seawater constituents on the cement hydration products, alkali-aggregate expansion, crystallization pressure of salts within the concrete matrix, frost action in cold climates, corrosion of embedded steel in reinforced or prestressed members, and physical erosion due to wave action and floating objects. Attacks on concrete tend to increase the concrete permeability, which in turn makes the concrete material progressively more susceptible to further action by the same destructive agents, as well as to other types of attack (*1*).

Permeability has been regarded by experts in the field to be a fundamental material property governing the durability of concrete, particularly in structures exposed to a marine environment (*2,3*). Mehta (*2*) reviewed the case histories of several concrete structures that had exhibited deterioration after long-term exposure to seawater. These case histories clearly showed that although physical and chemical interactions between seawater and constituents of portland cement took place, serious deterioration did not occur unless seawater was able to penetrate the interior of the concrete. The author concluded that "permeability rather than the chemistry of concrete was thus identified as the most important factor in long-term durability" (*3*). The author pointed out that structural cracking does not lead to corrosion as long as the remaining concrete is impermeable, especially if the concrete is fully submerged. Mehta also stated in his conclusions that harmful chemical attack can be limited to the surface by rigorous implementation of well-known measures to ensure low permeability of concrete.

Since the durability of concrete in a marine structure is greatly affected by its permeability, it is logical that the permeability of the in-service concrete should be used to assess its durability. However, currently there is no convenient and effective method that can be used for this purpose. There is a great need to develop such a method. This paper presents the research work conducted at the University of Florida to address this area of need.

Department of Civil Engineering, University of Florida, Gainesville, Fl. 32611.

OBJECTIVES OF THE STUDY

The primary objectives of this research study were the following:

1. To develop a field permeability test (FPT) apparatus and method for reliably and conveniently determining the permeability of in-service concrete;
2. To test and evaluate the developed FPT apparatus and method and establish, through laboratory and field experimentation, an effective and efficient testing procedure;
3. To implement the FPT apparatus and method in the testing of existing marine structures to determine the in situ permeability of concrete under actual field conditions; and
4. To investigate and attempt to establish any relationships that may exist between the results of the rapid chloride permeability tests and the results of the FPTs.

The development and evaluation of the FPT apparatus and method as stated in Objectives 1 and 2 have been presented in detail previously (*4*); therefore, they will be only briefly summarized in this paper. The work pertaining to Objectives 3 and 4 is presented in greater detail.

RESEARCH METHODOLOGY AND TESTING PROGRAM

The testing and analysis program of this study include the following major tasks:

1. The developed prototype FPT apparatus was tested in both the laboratory and actual field installations to evaluate its performance characteristics, to ensure that it is operating properly and satisfactorily, and to establish an effective testing procedure.
2. FPTs were run at various locations on in-service concrete bridges situated at or near the coast of Florida. In situ measurements of moisture content of the tested concrete were also made. Relevant design and construction information on the tested structures were obtained, when available.
3. Rapid chloride permeability tests were performed on concrete samples that were cored from selected test locations.
4. A comparative analysis of the experimental data obtained from the field and laboratory tests was performed to establish possible relationships among the results from the various tests.

FIELD TESTING PROGRAM

Selected Concrete Bridge Structures

In-service marine concrete structures, mainly bridges that are located at or near the coastal regions of Florida, are classified as being exposed to extremely aggressive or highly corrosive environments and thus were targeted for testing for a "worst-case scenario." The selection of these structures was primarily based on current needs and accessibility of the test sites. An attempt was made to test a variety of bridge types ranging from high-profile monumental bridge structures to small highway bridges. This variety offered the opportunity to test and evaluate concrete bridge structures of various ages, designs, and functions and utilizing various materials and construction methods. The following 13 concrete bridge structures in Florida were selected and tested: in the Florida Keys, Seven-Mile Bridge, Long Key Bridge, Bahia Honda-Southbound Bridge, Bahia Honda-Northbound Bridge, Spanish Harbor Bridge, Niles Channel Bridge, and Niles Channel Old Bridge; B. B. McCormick Bridge, Jacksonville; SR-206 Bridge, Crescent Beach; Seabreeze Causeway Bridge, Daytona Beach; Broward River (SR-105) Bridge, Jacksonville; Horse Creek Bridge and Horse Creek Old Bridge, Melbourne.

Six of these bridges are located on the Florida Atlantic Intracoastal Waterway (FAIW), whereas the other seven are located in the Florida Keys. The majority of these bridges were designed for a service life of 50 years, although the Seven-Mile and the Long Key bridges were designed for a life of 75 years.

A Florida Class IV concrete, specified to have a maximum allowable water–cement ratio of 0.41, and epoxy-coated reinforcing steel bars were used in the construction of the Seven-Mile, Long Key, and Niles Channel bridges. Cathodic protection(CP) systems were installed on a total of 20 pile columns of the B. B. McCormick Bridge and on the east-end pile caps (footers) of the SR-206 bridge. Conductive coating was applied to 60 piles and conductive concrete applied to 10 piles of the Seabreeze Causeway Bridge; conductive coating was also applied to the underdecking beams and end walls of the Horse Creek Bridge and to approximately 45 piles of the Broward River Bridge. The majority of these CP systems were installed and energized after concrete deterioration was detected and were used as remedial systems to reduce further corrosion.

Figures 1 and 2 show the extent and severity of concrete deterioration of some of the in-service concrete bridge structures tested in this study. Concrete cracking due to corrosion of the embedded steel reinforcement and subsequent spalling of concrete cover is the most predominant type of concrete deterioration encountered in marine concrete bridge structures in Florida. Figure 1 shows a severely deteriorated pile from the B. B. McCormick Bridge substructure located on the FAIW in Jacksonville. Extensive vertical cracking of concrete is seen from the steel corrosion. The corrosion of the embedded steel reinforcement in the pile is in an advanced state and the steel can readily be seen through the gap created in the material because of localized loss of concrete cover. This is a typical pattern of concrete distress that is demonstrated in the form of continuous vertical cracking running parallel to the edge of the pile and extending several feet above the high-tide level into the splash zone. The tidal zone,

FIGURE 1 Excessive vertical cracking and spalling of concrete in a pile (B. B. McCormick Bridge, Jacksonville).

defined as the region between the low- and high-tide levels, can easily be identified by the existence of various formations of marine organisms that extend to the high-tide level, above which the splash zone begins.

The extent and severity of concrete deterioration of in-service bridge structures in Florida are clearly manifested in Figure 2, which shows a massive structural element of the Seven-Mile Bridge substructure located in the Florida Keys, which is considered to be an extremely aggressive (highly corrosive) environment. Corrosion of the embedded steel reinforcement in this case is in the most advanced state in which decomposition of the material and disbondment from the surrounding concrete are accompanied by a complete loss of concrete cover, which in turn can lead to severe structural damage. In this particular pier, the concrete deterioration extends to about 9 ft above the tidal zone. This is contrary to the general observation that the most severe deterioration is likely to take place in the tidal zone where the concrete is exposed to various types of physical and chemical forces. As demonstrated in Figure 2, concrete deterioration caused by corrosion or other chemical and physical attacks is not a phenomenon that is encountered only in the tidal zone and that can readily be repaired or treated locally; it is a more extensive problem that has the potential to cause severe structural damage.

FIGURE 2 Excessive steel corrosion and extensive concrete spalling in a pier (Seven-Mile Bridge, Florida Keys).

Field Permeability Tests

The field testing operations included (a) site selection, (b) preparation of test location, (c) determination of size and location of the embedded reinforcing steel, (d) coring of test holes for FPT, (e) extraction of concrete cored samples for chloride permeability testing, (f) determination of moisture content of site concrete, (g) performance of FPTs, and (h) patching of cored concrete.

In situ permeability tests were performed on the selected concrete bridges using the developed FPT apparatus (*4,5*). FPTs were performed on structural elements such as partially submerged piling, pile columns, piers, pile caps, footers, bascules, and so forth, at various locations, elevations, and orientations. The majority of the structural elements tested were part of the substructure of these concrete bridges and were accessible only by water. Therefore, the flow measurements for the in situ permeability tests were taken remotely by means of the FPT instrumentation unit that was carried on and operated from a boat.

In situ permeability tests were run at several locations on the bridge substructures. Tests were run on the undeteriorated portion of a generally deteriorated element. Thus, the tested concrete was representative of the concrete material of the member. The surface of in situ concrete at the prepared test sections was cleaned of any marine organisms or other irregularities if the selected test location was within the tidal or splash zone. The FPTs were performed by inserting the FPT probe in a $\frac{7}{8}$-in.-diameter, 6-in.-deep hole drilled perpendicular to the concrete surface, sealing off the middle section of the test hole by means of the double-packer mechanism of the probe, and applying high pressure (150 to 500 lb/in.^{2} of gravity) to force the water to permeate radially into the concrete mass, as shown in Figure 3. The rate of flow of water into the test section (i.e., the injection rate) was constantly monitored by a manometer attached to the FPT instrumentation unit, which also contained the central control panel with the appropriate hydraulic quick-connections, flow valves, pressure regulator, and test gauge. Flow measurements were taken at regular intervals for as long as it was required to

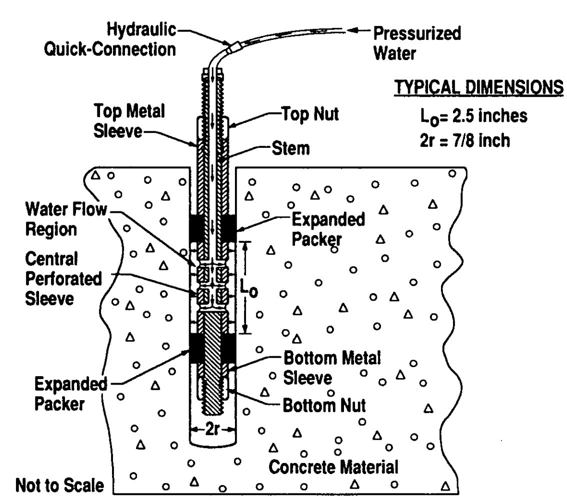

FIGURE 3 Schematic diagram of the FPT probe and setup.

reach a steady-state flow rate condition. The apparent coefficient of permeability of the concrete was determined from the steady-state flow rate by means of the following equations:

$$K = \frac{Q}{2\pi L_0 h} \sinh^{-1}\left(\frac{L_0}{2r}\right) \quad \text{for } r \leq L_0 < 10r \tag{1}$$

or

$$K = \frac{Q}{2\pi L_0 h} \ln\left(\frac{L_0}{r}\right) \quad \text{for } L_0 \geq 10r \tag{2}$$

where

K = coefficient of permeability (in./sec or cm/sec);
Q = steady-state flow (in.^{3}/sec or cm^3/sec);
h = applied pressure head (in. or cm);
$2r$ = diameter of test hole (in. or cm); and
L_0 = length of test section [in. or cm (see Figure 3)].

The derivation of Equations 1 and 2 can be found elsewhere (*4,5*). These equations are usually referred to as the Packer/Lugeon equations and are the ones used by the U.S. Bureau of Reclamation for in situ determination of rock mass permeability. For the FPT setup used, the length of the test section, L_0, was always between r and $10r$; thus, Equation 1 was used in computing the coefficient of permeability in an FPT.

The relative moisture content of the concrete site at each test location was measured before, during, and after testing by means of a nondestructive portable moisture meter at four diagonal points around the test hole. Since the accuracy of the moisture meter had not been investigated, the main purpose of using the moisture meter was to determine the relative change in moisture content during the test rather than to measure the moisture content of the concrete accurately. If the concrete was relatively dry, a vacuum preconditioning and subsequent presaturation of the test section were applied for approximately 30 min before any permeability testing. The concrete test section was saturated to the extent that further saturation would not affect the results of the FPT. It is estimated that in this condition the concrete was fully saturated to a minimum depth of 0.4 in. (1 cm) from the wall of the test hole.

The FPTs were conducted by one person and an assistant; test measurements were completed usually within 2 hr from the commencement of the actual test run. The entire FPT operation, including coring, preconditioning, testing, and patching of concrete, was completed within approximately 2 to 3 hr. The FPT apparatus demonstrated satisfactory performance; no major problems were encountered in any of the field installations.

LABORATORY TESTING PROGRAM

The standard test method for rapid determination of the chloride permeability of concrete (AASHTO T277-83) was employed in this study to further evaluate the concrete material of the in-service bridge structures under investigation. Cored concrete samples were obtained from selected test locations where FPTs were performed. However, to minimize the disruption to the concrete structures tested, 1.75-in.-diameter cores instead of the standard 3.75-in.-diameter cores were obtained. Figure 4 shows an actual concrete core obtained from one of these test locations as well as a prepared specimen as used in the chloride test. Since the 1.75-in. actual diameter of these cores was smaller than the standard 3.75-in.-diameter normally used in this test, the dimensions of the inside diameter of the applied voltage cell were modified to fit the outside diameter of the prepared field specimens. Figure 5 shows the modified applied voltage cell that was fabricated and used in this study.

During the preliminary testing of some of the cores using the modified applied voltage cell, temperatures as high as 180°F were observed within the first 2 hr of test duration. Thus, an additional modification was made to the rapid chloride test. The level of applied voltage was reduced from the standard 60 V to 30 V to avoid equipment failure caused by the observed higher-current flows through the tested concrete samples.

Because of these modifications, the results obtained from the rapid chloride permeability test were adjusted accordingly to take into account the reduced cross-sectional area of the concrete specimens and the reduced applied voltage. The test measurements were normalized to correspond to standard

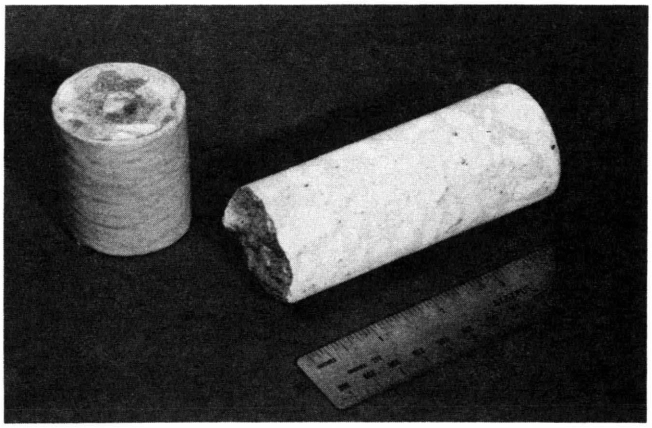

FIGURE 4 Field concrete core (*right*) and prepared specimen (*left*).

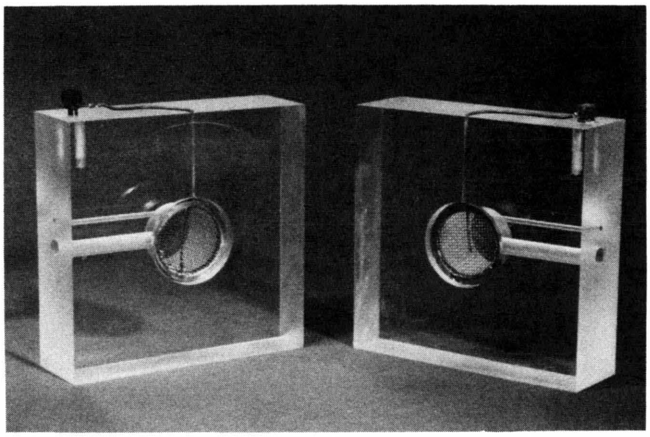

FIGURE 5 Modified applied voltage cell.

results by applying an adjustment factor C_a, determined as follows.

The electrical resistivity ρ of any material is defined as the resistance between opposite faces of a unit specimen of the material according to the following equation:

$$\rho = RA/L \tag{3}$$

where

R = electrical resistance of specimen,
A = cross-sectional area of specimen, and
L = length of specimen.

According to Ohm's law,

$$I = V/R \tag{4}$$

where I is the electrical current and V is the applied voltage. Since the length of the concrete specimens used in the chloride test device was constant (2 in.) and the resistivity of the concrete material during the testing was assumed to remain constant, the relationship between the standardized and modified chloride test results can be represented by the following simple equation:

$$I_{60}/I_{30} = V_{60}A_{60}/V_{30}A_{30} = C_a \tag{5}$$

where I_{60}, V_{60}, A_{60} are test parameters at the standard 60-V potential level and I_{30}, V_{30}, A_{30} are test parameters at the reduced 30-V potential level.

For the modified test setup used, the adjustment factor C_a was computed to be 9.18. This adjustment factor was used to normalize the test results obtained from the modified chloride test to correspond to the standard chloride permeability test results (in coulombs) as interpreted by AASHTO T277-83.

TEST RESULTS

FPT Results

Repetitive FPTs were performed on the same test sections to evaluate the repeatability of the test results. Representative results for various structural elements are displayed in Table 1. The data show that the measured permeability coefficients have a maximum percent difference ranging from about 3 to 28 percent. The results obtained from these replicate tests indicated that the FPT method demonstrated acceptable repeatability under actual field conditions. However, much higher variations were observed among the mean coefficients of permeability obtained from FPTs performed at various test locations on structural elements having the same design or concrete type, or both. Table 2 displays representative test results from FPTs performed at various test locations. As can be seen from these data, the maximum percent difference in such cases ranged between about 5 and 684 percent. The highest variation among the permeability coefficients of the same class of concrete was obtained from FPTs performed on three different test sections from a girder of the SR-206 Bridge at Crescent Beach, which was built in 1975. From a total of 57 individual FPTs run on the 13 selected concrete bridges, the lowest registered value of permeability coefficient, 0.94 × 10^{-9} cm/sec, was the one obtained from a test section on a V-pier cap of Pier 103 at the Long Key Bridge. Overall, the permeability coefficients of the tested site concretes were, on average, two to four orders of magnitude higher than the coefficients obtained from FPTs performed on laboratory-prepared concretes of similar quality (*5*,*6*).

TABLE 1 Results of FPTs Performed at the Same Test Section

CONCRETE BRIDGE STRUCTURE TESTED	TYPE OF STRUCTURAL ELEMENT	PERMEABILITY COEFFICIENT (xE-9 cm/sec) 1st TEST	RE-TEST	MAXIMUM PERCENT DIFFERENCE
SEVEN MILE	PIER STRUT	1.90	1.85	2.70
	"	6.69	8.09	-17.30
BAHIA HONDA - NORTHBOUND	FOOTER CAP	5.25	6.21	-15.46
B. B. McCORMICK	PILE COLUMN	10.00	9.50	5.26
	"	45.70	35.60	28.37
	"	47.20	45.20	4.43
	ONSHORE PIER	41.70	34.80	19.83
		22.80	24.01	-5.04
SEABREEZE CAUSEWAY	BASCULE	25.80	20.80	24.04
	"	24.60	19.20	28.12

Chloride Permeability Test Results

Rapid chloride permeability tests were performed in the laboratory on a number of cored concrete specimens (1.75 in. in diameter) obtained from the selected bridge structures under investigation. The concrete cores used in this series of tests were obtained from structural elements that already had been tested for their relative water permeability by means of the developed FPT apparatus. These cores were obtained from locations as close as possible (within 12 in.) to the tested FPT sections.

The field-cored concrete specimens were sectioned in 2-in. slices and subsequently conditioned and prepared for testing according to the procedure stipulated by AASHTO T277-83. As many as three 2-in. slices were cut from the top (A), middle (B), and bottom (C) sections of long field cores, when possible. The middle (B) section of concrete cores, corresponding to the region of concrete material tested with the FPT, was used in these tests for comparative purposes. The test measurements were analyzed according to the standard procedure, and the adjustment factor C_a was applied to convert the chloride permeability results (in coulombs) to correspond to standard values. The results from this series of laboratory chloride permeability tests are presented in Table 3.

The maximum percent difference between the values of the charge passed (in coulombs) of replicate test specimens taken from the top, middle, and bottom sections of field concrete cores ranged between about 1 and 242 percent. A relatively high variation was also observed among the values of structural elements having the same design or concrete class, or both, as in the case of the water permeability results. In 80 percent of the cases, the relative chloride permeability (in terms of coulombs) of the tested concrete material was higher for concrete specimens cut from the bottom or the middle sections of cores as compared with those cut from the top. Although a definite relationship quantifying this effect was not achieved during this research study, this trend, along with the discrepancies in the test results obtained for the same class of concretes, indicated a significant variation in the quality and durability characteristics of the tested concretes. One possible explanation for the high variation of the results of the rapid chloride permeability tests is that they are affected by the chloride ion contents of the test concrete. Since the cores contained variable amounts of chloride ions, high variations in the test results were obtained.

TABLE 2 Results of FPTs Performed on Various Structural Elements of the Same Type and Concrete Material

CONCRETE BRIDGE STRUCTURE TESTED	TYPE OF ELEMENT TESTED	TEST SECTION	MEAN COEFFICIENT OF PERMEABILITY (xE-9 cm/sec)	MAXIMUM PERCENT DIFFERENCE
SEVEN MILE	STRUT	1	1.875	294.13
	"	2	7.390	
LONG KEY	PIER CAP	1	0.940	162.77
	"	2	2.470	
	"	3	2.360	
BAHIA HONDA - SOUTHBOUND	FOOTER	1	7.210	533.38
	"	2	27.760	
	"	3	6.650	
	"	4	42.120	
BAHIA HONDA - NORTHBOUND	"	1	5.730	550.09
	"	2	23.930	
	"	3	37.250	
SPANISH HARBOR	PILE	1	3.880	6.89
	"	2	3.630	
NILES CHANNEL	FOOTER	1	1.460	21.92
	"	2	1.780	
NILES CHANNEL - OLD	WALL	1	51.030	30.41
	"	2	66.550	
B. B. McCORMICK	PILE	1	9.750	470.37
	"	2	40.070	
	"	3	46.200	
	"	4	14.500	
	"	5	19.800	
	"	6	8.100	
	"	7	37.730	
	"	8	23.400	
SR206 - CRESCENT BEACH	GIRDER	1	12.000	684.31
	"	2	1.530	
	"	3	5.650	
SEABREEZE CAUSEWAY	BASCULE	1	23.300	109.91
	"	2	11.100	
	"	3	21.300	
BROWARD RIVER	PILE	1	8.030	304.73
	"	2	32.500	
HORSE CREEK	BEAM	1	10.640	4.51
	"	2	7.970	
	"	3	11.120	
HORSE CREEK - OLD	WALL	1	5.910	13.87
	"	2	6.730	

Correlation Between the Results of the FPT Method and AASHTO T277-83

The results obtained from the developed FPT method and the standard AASHTO T277-83 were compared to identify any possible relationships between them. Since the center of

TABLE 3 Results of Rapid Chloride Permeability Tests

CONCRETE BRIDGE	TYPE OF ELEMENT	SPECIMEN No.	CHARGE PASSED (COULOMBS) ACTUAL (M)	ADJUSTED (S)	MAX. % DIFF.
SEABREEZE CAUSEWAY	BASCULE	1B	420.2	3,857.4	N/A
		2B	822.2	7,547.9	N/A
B. B. McCORMICK	PILE	3B	372.8	3,422.3	N/A
	PIER	4B	289.6	2,658.9	N/A
BROWARD	BASCULE	5B	209.3	1,921.7	N/A
OLD HORSE CREEK	WALL	6A	230.0	2,111.8	0.76
	"	6B	231.8	2,127.9	
	"	7A	344.0	3,158.2	N/A
	"	8B	211.1	1,937.8	127.45
	"	8C	480.1	4,407.5	
NEW HORSE CREEK	BEAM	9A	298.0	2,735.3	178.68
		9B	572.1	5,252.2	
		9C	830.4	7,622.7	
		10A	251.8	2,311.0	242.02
		10B	861.0	7,904.1	
NILES CHANNEL	STRUT	11A	637.9	5,855.6	35.02
	"	11B	861.2	7,906.2	
OLD NILES CHANNEL	WALL	12A	2000.9	18,368.1	21.22
	"	12B	1650.7	15,153.1	
	"	13B	1985.6	18,227.6	N/A
SP.HARBOR	PILE	15A	1139.2	10,457.5	N/A
BAHIA HONDA SOUTH BOUND	FOOTER	16B	1002.8	9,205.7	9.79
	"	16C	1101.0	10,107.2	
	"	17B	1240.7	11,390.0	46.83
	"	17C	845.0	7,757.5	
CONCRETE BRIDGE	TYPE OF ELEMENT	SPECIMEN No.	CHARGE PASSED (COULOMBS) ACTUAL (M)	ADJUSTED (S)	MAX. % DIFF.
BAHIA HONDA NORTH BOUND	FOOTER	18B	845.2	7,758.6	46.29
	"	18C	1236.4	11,350.1	
	"	19A	863.4	7,926.2	26.22
	"	19B	1089.8	10,004.4	

Key:

(M) = Test results using the Modified Applied Voltage Cell with a reduced 30-volt potential level

(S) = Estimated equivalent charge corresponding to coulomb values as interpreted in the Standard AASHTO T277-83 method

N/A = Not Applicable

Note:

According to the standard AASHTO T277-83 method:

Charge Passed (coulombs)	Chloride Permeability
>4,000	High
2,000-4,000	Moderate
1,000-2,000	Low
100-1,000	Very Low
<100	Negligible

the flow region in a typical FPT was located about 3 in. from the surface of the concrete, the chloride permeability test results obtained from the specimens cut from the middle of concrete cores (B-series) corresponding to the FPT section were used for comparison. The comparison of the results from these two tests is presented in Table 4.

A linear regression analysis between these two variables was performed. The results of the regression analysis were as follows:

$$Q_{RCP} = 247.7167 K_{FPT} + 1834.190$$

$$R^2 = 0.90$$

$$\text{Standard Error of the Slope} = 22.01 \qquad (6)$$

TABLE 4 Comparison of Results Obtained from the FPT and Rapid Chloride Permeability Test

CONCRETE BRIDGE STRUCTURE TESTED	TYPE OF STRUCTURAL ELEMENT	CHLORIDE PERMEABILITY B-SERIES (COULOMBS)	MEAN WATER PERMEABILITY COEFFICIENT (xE-9 cm/sec)
SEABREEZE CAUSEWAY	BASCULE	3,857.4	11.10
	"	7,547.9	21.30
B. B. McCORMICK	PILE	3,422.3	9.75
	PIER	2,658.9	8.10
BROWARD RIVER	BASCULE	1,921.7	6.08
HORSE CREEK - OLD	WALL	2,127.9	5.91
	"	1,937.8	6.73
HORSE CREEK - NEW	BEAM	5,252.2	10.64
	"	7,904.1	7.97
NILES CHANNEL	STRUT	7,906.2	14.70
NILES CHANNEL - OLD	WALL	15,153.1	51.03
	"	18,227.6	66.55
BAHIA HONDA - SOUTHBOUND	PIER. FOOTER	9,205.7	27.76
	"	11,390.0	42.12
BAHIA HONDA - NORTHBOUND	"	7,758.6	23.93
	"	10,004.4	37.25

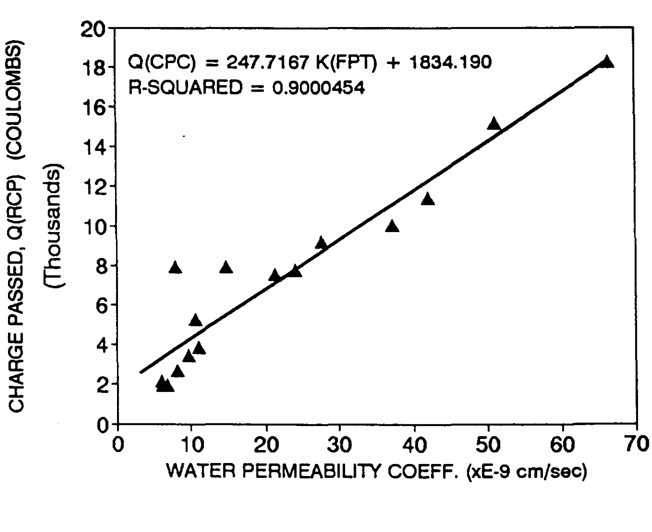

FIGURE 6 Correlation between charge passed and water permeability coefficient.

where Q_{RCP} is the charge passed (in coulombs) measured by the rapid chloride permeability (RCP) test method and K_{FPT} is the coefficient of permeability as determined by the FPT method. Figure 6, which is a graphical representation of these results, suggests that it is possible to assume a linear relationship between the respective chloride and water permeability values as determined by the two methods. Although limited experimental values were considered in this analysis, the obtained coefficient of determination (R^2) of 0.90 indicates a strong correlation between the two quantities.

DISCUSSION OF RESULTS

The evaluation of the in-service concrete structures investigated in this study was performed from the concrete material point of view and with respect to the physical property of permeability. Comparison of the results obtained from FPTs performed on the selected in-service concrete structures with those obtained from water permeability tests on laboratory-prepared concrete specimens of similar quality and materials indicates that the site concretes exhibited very high water permeabilities (*6–8*).

According to the notes in the standard AASHTO T277-83 procedure (*9*), conventional portland cement concrete of low *w/c* ratio (less than 0.4), which corresponds to the Florida Class IV concrete used in some of the previously mentioned bridge structures, typically is expected to have low chloride permeability corresponding to a total charge passed ranging between 1,000 and 2,000 coulombs, as indicated in Table 3. However, only 7 percent of the tested concrete specimens produced results that were within this range, whereas 28 and 66 percent of the specimens exhibited moderate and high chloride permeabilities, respectively. All of the concrete specimens extracted from bridge structures located in the Florida Keys region exhibited high chloride permeabilities. The highest registered charge passed through a tested concrete material—18,368 coulombs—was obtained from a concrete core extracted from the arch wall at the Old Niles Channel Bridge. This value was about 360 percent higher than the upper limit of charge passed corresponding to high chloride permeability according to AASHTO T277-83. This bridge was built in 1906, and the concrete material at the test location was determined to have a chloride content of approximately 100 lb/yd^3 at the time the FPTs were performed (*8*). Based on the high quantities of charge passed through the majority of the tested concretes and the observed high variation of values obtained from duplicate specimens tested with the rapid chloride permeability method, could indicate that there was a significant variation in the quality and durability characteristics of the concrete with respect to chloride permeability.

Although the site concrete was sound both at the particular locations where the FPTs were performed and where cored samples were extracted and did not exhibit any visual signs of deterioration, the results obtained from the field and laboratory testing programs indicated poor durability characteristics with respect to permeability. The results further suggest that there seems to be an apparent relationship between the condition of structural concrete and its permeability. Concrete bridge structures that demonstrated durability problems also exhibited high permeability of the concrete material. For example, the permeability values obtained from a number of piles from B. B. McCormick Bridge, which has exhibited severe concrete deterioration (Figure 1), were substantially higher than those obtained from the piles from Horse Creek Bridge, which has shown no concrete deterioration (Table 2). It is believed that the relatively high permeability of the concrete contributed significantly to the severe deterioration of some elements in these bridge structures by allowing the intrusion of deleterious substances present in the marine environment into the concrete.

CONCLUSIONS

In this study, an FPT apparatus and method were developed and evaluated in both the laboratory and the field. The developed prototype FPT apparatus and method were used in the testing and evaluation of in-service marine structures in conjunction with the standard rapid chloride permeability test and laboratory water permeability test.

On the basis of the experimental findings of the field and laboratory testing programs, the major conclusions of this research study are summarized as follows:

1. The developed FPT apparatus and method appear promising in providing a suitable measuring system for the rapid, convenient, and reliable determination of the in situ water permeability of structural concrete. The repeatability of the FPT results under actual field environment was shown to be acceptable.
2. There appears to be a linear relationship between the charge passed (coulombs) through a concrete material as measured by the rapid chloride permeability test (AASHTO T277-83) and its corresponding water permeability, with a coefficient of determination, R^2, of 0.90.
3. There appears to be a relationship between permeability and durability of concrete in service. The concrete material that exhibited durability problems also demonstrated high permeability.
4. The deterioration of concrete in the tested bridge structures was primarily caused by corrosion of the embedded steel reinforcement, which was caused by the intrusion of chloride ions. It is believed that the high permeability of the in situ concrete material significantly contributed to this effect.
5. A high variation in permeability was observed between concrete bridge structures specified to have the same concrete class. Since permeability is greatly affected by the microstructure of the concrete, high variations in permeability test results are usually expected, especially for in-service concrete. It is recommended that a sufficient number of tests be performed and that the data be statistically analyzed to account for such variations.
6. The FPT method can provide a relative measurement of permeability that can be used as an indication of the quality and performance characteristics of structural concrete.

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REFERENCES

1. P. K. Mehta. *Concrete, Structures, Properties, and Materials*, Chapter 5. Prentice-Hall, Inc., Englewood Cliffs, N.J., 1986, pp. 105–169.
2. P. K. Mehta. Durability of Concrete in Marine Environment—A Review. *Performance of Concrete in Marine Environment*, SP-65. American Concrete Institute, Detroit, Mich. 1980, pp. 1–20.
3. P. K. Mehta. Durability of Concrete Exposed to Marine Environment—A Fresh Look. *Concrete in Marine Environment*, SP-109. American Concrete Institute, Detroit, Mich. 1988, pp. 1–23.
4. C. A. Meletiou, M. Tia and D. Bloomquist. Development of a Field Permeability Test Apparatus and Method for Concrete. *American Concrete Institute Materials Journal*, Vol. 89, No. 1, Jan.–Feb. 1992, pp. 83–89.
5. C. A. Meletiou. *Development of a Field Permeability Test for Assessing the Durability of Concrete in Marine Structures*. Ph.D. dissertation, University of Florida, Gainesville, 1991.
6. M. Tia et al. *Field and Laboratory Study of Modulus of Rupture and Permeability of Structural Concretes in Florida*. Final Report, UF Project 491-450423412. Department of Civil Engineering, University of Florida, Gainesville, Aug. 1990.
7. P. Soongswang, M. Tia, D. Bloomquist, C. A. Meletiou, and L. Sessions. An Efficient Test Set-Up for Determining the Water-Permeability of Concrete. In *Transportation Research Record 1204*, TRB, National Research Council, Washington, D.C., 1988, pp. 77–82.
8. P. Soongswang, M. Tia, and D. Bloomquist. Factors Affecting the Strength and Permeability of Concrete Made with Porous Limestone. *ACI Materials Journal*, Vol. 88, No. 4, July–Aug. 1991, pp. 400–406.
9. Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete. AASHTO T277-83. In *Standard Specification for Transportation Materials and Methods of Sampling and Testing*, 14th ed. AASHTO, Washington, D.C., 1986.
10. *Concrete Chloride Content Ledger*, Vol. 1. Florida Department of Transportation, March 1978–July 1981.

DISCUSSION

KENNETH A. SNYDER
National Institute of Standards and Technology, Gaithersburg, Md. 20899.

I would like to present several points for review by the authors.

1. I feel that a key element in the presentation was the omission of laboratory water permeability measurements performed on the field specimens. I acknowledge that laboratory tests were performed to correlate the results of the portable permeability meter (PPM) and standard laboratory water permeability tests. I would agree that the results were very encouraging. However, field tests performed on 3-ft-diameter columns are more suspect to experimental problems than tests performed on 6- × 6- × 12-in. specimens; I am only guessing at the size of specimens on the basis of the presentation slides.

The primary concern is saturation. I would venture to guess that the 6- × 12-in. prisms were properly cured in saturated lime water. Therefore, the entire life of the specimen was spent submerged, and, hence, fully saturated. I don't believe that "vacuum saturating" a 3-ft-diameter column for one-half hour is sufficient to ensure saturation, not to mention an extreme experimental hurdle to be performed correctly.

2. The authors' experimental apparatus contains a feature that leads to skepticism of the results. When the fluid in the middle region is pressurized, the greatest pressure gradient exists between the middle region and the two regions in the cored hole immediately above and below the middle pressurized region. This suggests that the immediate tendency of the fluid would be to flow back into the core. This behavior is different from the spherical geometry the authors mentioned in the presentation. If no fluid is found in the regions immediately above and below the middle region then there could be additional shortcomings of the assumptions used in the permeability calculation. The lack of fluid would indicate that the fluid does not penetrate the concrete very far, suggesting a cylindrical geometry rather than a spherical one.

3. I would also like to discuss the mathematical development of the authors' equations to calculate permeability from the flow. I realize that the nature of TRB dissuades from discussing involved mathematical development and, therefore, I realize that my questions already may have been addressed in a technical publication.

There was no mention of a time dependence to the solution of the permeability calculation. I believe you mentioned that spherical symmetry was assumed along with an infinite medium. If one assumes an infinite medium, then the pressure gradient must be zero to satisfy the boundary conditions and, hence, there should be no flow. Practically, one accepts that there must be a pressure gradient at the core wall the instant the pressure is applied. However, as fluid moves out into the concrete the "rising edge" of the pressure gradient must be moving out also. This should continue until the rising edge reaches the outside surface of the object under test. All the while, the flow from the probe would slowly diminish because of the changing pressure gradient. I believe that the authors simply stated that the flow was recorded until a steady state was reached.

4. An additional aspect of the test that should be considered is the extent of the test. Consider the depth to which the fluid (I assume water) penetrates the system, which can be roughly approximated from the following calculation. The volume-averaged Darcy equation can be expressed in differential form:

$$\langle \mathbf{v} \rangle = -\frac{k}{\mu} \nabla \langle \mathbf{p} \rangle$$

Consider that the closest region of atmospheric pressure is just a few centimeters away above and below the probe's middle section. Assuming an applied pressure of 100 lb/in.^{2} and a reasonably permeable sample, the velocity of the fluid penetrating the concrete would still be very small:

$$\langle \mathbf{v} \rangle = \frac{10^{-16}\text{m}^2}{10^{-3}\text{ kg/m sec}} \frac{7 \times 10^5 \text{ N/m}^2}{2 \times 10^{-2}\text{m}} = 4 \times 10^{-6} \text{ m/sec}$$

Over the course of 1 hr the fluid would advance only approximately 1.5 cm. Since this is the greatest pressure gradient the fluid would experience, it is an upper limit of the maximum penetration.

5. The oil industry is very interested in determining the permeability of underground rock formations to estimate, among other things, the productivity of a potential oil well. The development of a meter to measure the in situ permeability holds a keen interest for them. When one considers the immeasurable person hours and dollars spent by petroleum engineers trying to develop such a device, which has still eluded them, it is a good idea to look with a critical eye at any new developments of such a device.

AUTHORS' CLOSURE

The development of the FPT apparatus and method used in this study has been presented in detail in a previous paper by the authors (*1*); the answers to most of the questions raised by the discussant can be found therein. The derivation of the analytical equation relating the flow rate measured in the FPT and the water permeability of the concrete is presented as follows.

It is assumed that the mass concrete is infinite in size and that a steady-state flow condition has been reached.

The following parameters are defined:

R = radius of influence (i.e., the effective radius of the flow region under study) and

$\Phi(R)$ = the potential at a distance R from the center of the sphere.

The discharge, Q, at any radial distance ρ from the source at the center of the sphere is

$$Q = 4\pi\rho^2 v_\rho = 4\pi\rho^2 \frac{\partial \Phi}{\partial \rho}$$

$$\frac{\partial \Phi}{\partial \rho} = \frac{Q}{4\pi\rho^2} \qquad (1)$$

where $4\pi\rho^2$ is the surface area of the sphere and v_ρ is the radial velocity.

By integrating Equation 1,

$$\int \partial \Phi = \int \frac{Q}{4\pi\rho^2}\, \partial \rho$$

we obtain the potential, Φ, as

$$\Phi = -\frac{Q}{4\pi\rho} + C \qquad (2)$$

Equation 2 indicates that in the case of spherical flow the potential varies inversely with the radius of influence.

To determine the constant of integration, C, in Equation 2, the boundary conditions at the hole should be defined as follows: $\rho = r$ and $\Phi = \Phi_0$. Therefore, by solving Equation 2 with respect to C we get

$$C = \Phi_0 + \frac{Q}{4\pi r} \qquad (3)$$

By applying the boundary conditions and substituting the expression for C in Equation 3 into Equation 2, we get

$$\Phi = \frac{Q}{4\pi}\left(\frac{1}{r} - \frac{1}{\rho}\right) + \Phi_0 \tag{4}$$

where $r \leq \rho \leq R$.

The following parameters are also defined: $2r$ = diameter of test hole and $L_0 = 2L$ = length of test section. According to Girinsky (2), the discharge, Q, can be assumed to be constant along the test section, and the discharge per unit length can be expressed as

$$\frac{dQ}{d\eta} = \frac{Q}{2L} \Rightarrow dQ = \frac{Q}{2L}\, d\eta \tag{5}$$

Assuming Equation 2 to represent the potential at any point in the flow region,

$$d\Phi = \frac{dQ}{4\pi\rho} \tag{6}$$

For a cylindrical coordinate system,

$$\rho = [r^2 + (z - \eta)^2]^{1/2} \tag{7}$$

By combining Equations 5 through 7 we get

$$d\Phi = \left(\frac{Q}{2L}\right) d\eta \left(\frac{1}{4\pi\rho}\right) \Rightarrow d\Phi = \left\{\frac{Q}{8\pi L[r^2 + (z - \eta)^2]^{1/2}}\right\} d\eta$$

Set the limits of integration from $(L_0/2)$ to $+ (L_0/2)$ (note that $L_0 = 2L$):

$$\Phi(r, z) = \frac{Q}{8\pi L}\int_{-L}^{+L} \frac{d\eta}{[r^2 + (z - \eta)^2]^{1/2}} \tag{8}$$

$$\Rightarrow \Phi(r, z) = \frac{Q}{8\pi L}\left[\sinh^{-1}\left(\frac{z + L}{r}\right) - \sinh^{-1}\left(\frac{z - L}{r}\right)\right] \tag{9}$$

According to Harr (3), the equipotential surfaces given by Equation 9 are seen to be ellipsoids.

By assuming $z = 0$ and substituting $L_0 = 2L$, Equation 9 becomes

$$\Phi(r) = \frac{Q}{2\pi L_0}\sinh^{-1}\left(\frac{L_0}{2r}\right) \approx \frac{Q}{2\pi L_0}\ln\left(\frac{L_0}{r}\right) \tag{10}$$

where

$$\sinh^{-1}(x) = \ln[x + (x^2 + 1)^{1/2}]$$

and if $x >> 1$, $\sinh^{-1}(x)$ is approximately equal to $\ln(2x)$.

Finally, if we substitute in Equation 10 $Kh = \Phi$ and solve with respect to the permeability K, we obtain

$$K = \frac{Q}{2\pi L_0 h}\sinh^{-1}\left(\frac{L_0}{2r}\right) \quad \text{for } r \leq L_0 < 10r \tag{11a}$$

and

$$K = \frac{Q}{2\pi L_0 h}\ln\left(\frac{L_0}{r}\right) \quad \text{for } L \geq 10r \tag{11b}$$

where K is the coefficient of permeability and h is the applied pressure head.

Equations 11a and b relate the measured flow rate in the FPT to the coefficient of permeability of the concrete tested. For the FPT setup used, the length of the test section, L_0, is always between r and $10r$, and thus Equation 11a is used in computing the coefficient of permeability in the FPT.

The equation is based on the assumption that a steady-state flow condition has been reached. Thus, the recommended testing procedure requires that the FPT be run until an apparent steady-state flow condition has been reached. As stated in the paper, the concrete test sections were presaturated to the extent that further saturation would not affect the results of the FPT. It might be true that at the apparent steady-state condition, the test concrete had not been completely saturated and the flow had not reached its true steady-state condition. However, for practical purposes, since the test results do not change appreciably after that point, the determined permeability should be very close to the true permeability.

REFERENCES

1. C. A. Meletiou, M. Tia, and D. Bloomquist. Development of a Field Permeability Test Apparatus and Method for Concrete. *American Concrete Institute Materials Journal*, Vol. 89, No. 1, Jan.–Feb. 1992, pp. 83–89.
2. N. K. Girinsky. Determination of the Coefficient of Permeability." *Gosgeolizdat*, 1950.
3. M. E. Harr. *Groundwater and Seepage*. McGraw-Hill, Inc., 1962, pp. 259–262.

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