

SHRP-S-330

Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion

Volume 8: Procedure Manual

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Abstract

A procedure is presented for the condition assessment of concrete bridge components. It is based on a rational integration of thirteen applicable, current test methods or procedures with seven newly-developed methods. The selection of the current methods is detailed in Volume 1 of this report, and the development of the seven new procedures is presented in Volumes 2 through 7. While the primary emphasis is on deterioration associated with chloride-induced corrosion of reinforcing steel, all aspects of durability relative to concrete bridge components are addressed. The procedure is designed to be flexible and amenable to tailoring to suit the needs of the individual highway agency.

Executive Summary

The deterioration of the nation's concrete highway bridges is proceeding at an alarming rate. It has been estimated that the cost of this damage currently stands at about \$20 billion, and that it is increasing at the rate of \$500 million per year.

It is generally agreed that life-cycle cost analyses of viable alternatives are necessary in order to develop rational strategies for the repair, rehabilitation, and replacement of concrete bridge components. This, in turn, necessitates the acquisition of reliable information on the level and rate of deterioration.

This manual presents a procedure for the acquisition of critical data for assessing the condition of concrete bridge components. It incorporates thirteen currently used tests or procedures and seven new test methods. The thirteen present methods include 10 American Society of Testing and Materials (ASTM) standard test methods, one American Concrete Institute standard practice, a recently published test method for alkali-silica reactivity from the Strategic Highway Research Program, and the widely used method for measuring reinforcement cover using magnetic flux devices. The investigation that led to the choice of these current methods is covered in Volume 1 of this report.

The development of the seven new test methods is covered in Volumes 2 through 7 of this report. Detailed descriptions of the new test methods, in ASTM format, are presented in Appendices A through G of this volume.

The developed procedure consists of three major parts:

- Initial (baseline) evaluation survey;

- Subsequent evaluation surveys; and
- Evaluation surveys for special conditions..

The initial (baseline) evaluation survey consists of acquiring initial property data (e.g., compressive strength, permeability, air-void characteristics, etc.), which will provide a relative measure of initial overall quality. Most of this information is routinely collected at the time of construction, or shortly thereafter.

Subsequent evaluation surveys are carried out periodically to monitor the condition of the concrete bridge components. It is the data from these surveys, primarily, that define the condition and the rate of deterioration at any point in time.

Evaluation surveys for special conditions include asphalt-covered decks, pretensioned and post-tensioned prestressed concrete members, and rigid deck overlays.

The procedure is designed to be flexible in order to permit adjustments to accommodate the needs of individual highway agencies.

1

Introduction

Deterioration of Concrete Bridges

The deterioration of the nation's concrete highway bridges is proceeding at an alarming rate. Based on information provided by the Strategic Highway Research Program (SHRP) (1), it is estimated that the cost of this damage now stands at over \$20 billion, and that it is increasing at the rate of \$500 million per year.

The primary cause for deterioration of concrete bridge components is the chloride-induced corrosion of the reinforcing steel. Wedging action resulting from the production of voluminous corrosion reaction products causes cracking and spalling of the concrete. Normally, reinforcing steel is in a passive (non-corroding) state due to the highly basic environment ($\text{pH} > 12.5$) in concrete (2). However, the presence of chloride ions at the concrete/steel interface in excess of the reaction threshold level depassivates the steel, and corrosion usually ensues. On bridges, the source of the chloride ion is usually deicing chemicals applied in the snowbelt areas for winter trafficability or contact (directly or by means of aerosols) with seawater in coastal areas.

Other potential causes for deterioration of concrete bridge components include:

- Freezing and thawing damage due to inadequately air-entrained concrete or frost-susceptible aggregate materials;
- Alkali-silica and alkali carbonate reactivity for certain aggregate materials;

- Use of an excessive amount of water in the concrete mixture;
- Improper concrete placement, finishing, or curing practices;
- Accidental damage from collision or fire; and
- Design practices that fail to properly consider drainage requirements, stresses due to live and dead loads, shrinkage, or expansion.

The development of rational repair/rehabilitation/replacement strategies for bridges requires life-cycle cost analyses of viable alternatives. Reliable information on the conditions and rates of deterioration of the concrete bridge components is essential to this approach.

Purpose

This manual provides technical personnel of state highway agencies with information and specific directions for a detailed, coordinated procedure for the assessment of the condition of concrete bridge components. However, the ultimate purpose of obtaining the assessment data is to allow the formulation of rational decisions regarding repair, rehabilitation, and replacement of concrete bridge components. Therefore, in the larger sense, the purpose of this manual is to provide the guidelines for the acquisition of key information used in bridge management.

Scope

Even though the primary problem involved in the deterioration of concrete bridge components involves chloride-induced corrosion of reinforcement, this manual, for completeness, presents a procedural plan that covers deterioration from virtually all sources. On the other hand, the procedural plan provides for tailoring one's approach in order to accommodate the requirements of particular locales, materials, bridge types, or jurisdictional policies or priorities. The user has the option of deleting those aspects of the procedure that are irrelevant to his/her specific case.

2

Background

Basis

The developed procedure is based on the integration of selected current practices with new methods developed under SHRP project C-101. The topics of the developmental areas covered by project C-101 were specified by SHRP to cover known or perceived weaknesses in the data acquisition capability of the current technology relative to condition assessment of concrete bridge components. Thus, assuming that the validity of the original premise holds true and that suitable procedures have evolved from the developmental work of project C-101, all of the tools that are needed for a comprehensive procedure now exist.

Selected Current Practices and Methods

Based on a review of the technical literature and telephone interviews with maintenance and materials engineers in 47 states and 9 Canadian provinces, the best of currently employed testing procedures for evaluating conditions and causes related to concrete bridge component deterioration were identified. The details may be found in Volume 1, "State of the Art of Existing Methods," of this report series. In addition, an American Concrete Institute committee report procedure (ACI 224.1R) was added to fill a void relative to cracking (13). Also, a test procedure for alkali-silica reactivity, which was only recently developed and published under SHRP project C-202, was included (10). The resulting 13 methods are summarized in Table 2-1.

Table 2-1. Current practices adopted for bridge condition assessment procedure.

Subject	Standard Practice Designation	Reference
1. Reinforcement cover depth using covermeters (magnetic flux devices)	none	—
2. Concrete strength from test cylinders	ASTM C39	(3)
3. Concrete strength from drilled cores	ASTM C42	(4)
4. Concrete strength from pullout tests	ASTM C900	(5)
5. Concrete strength/quality indication from rebound hammer tests	ASTM C805	(6)
6. Concrete strength/quality indication from penetration tests	ASTM C803	(7)
7. Characteristics of the air-void system in hardened concrete	ASTM C457	(8)
8. Microscopic evaluation of the quality of hardened concrete (petrographic examination)	ASTM C856	(9)
9. Identification of alkali-silica reactivity	Proposed ASTM Standard ^a	(10)
10. Delamination detection by sounding	ASTM D4580	(11)
11. Damage assessment by pulse velocity	ASTM C597	(12)
12. Assessment of cracking	ACI 224.1R	(13)
13. Assessment of the probability of the existence of active reinforcement corrosion (half cell method)	ASTM C876	(14)

^aASR Handbook, SHRP-C/FR-91-101

New Procedures

Seven new test procedures were developed under SHRP project C-101. These are intended to cover perceived weak areas regarding equipment or methodology for the acquisition of data on concrete bridge component deterioration condition or rate. The development of these techniques is covered in Volumes 2 through 7 of this report series. The new test methods

are presented, in American Society for Testing and Materials (ASTM) format, in Appendices A through G of this report volume. Table 2-2 summarizes the new methods.

The twenty methods listed in Tables 2-1 and 2-2 constitute the tools that implement the procedure for the assessment of concrete bridge components set forth in the next chapter.

Table 2-2. Methods developed in SHRP C-101 included in bridge condition assessment procedure.

Subject	Test Procedure (Appendix)	Development Details (Report Volume)
1. Measuring the rate of corrosion of reinforcing steel in concrete	A	2
2. Assessing the condition of asphalt-covered bridge decks using pulsed radar	B	3
3. Determining the integrity of membrane systems on bridge decks	C	4
4. Determining the effectiveness of penetrating concrete sealers using the electrical resistance method	D	5
5. Determining the effectiveness of penetrating concrete sealers using the water absorption method	E	5
6. Determining the chloride content of concrete in the field	F	6
7. Determining the relative permeability of concrete in the field	G	7

3

Procedure

Initial (Baseline) Evaluation Survey

A flow diagram detailing the procedure developed for assessing the condition of concrete bridge components is presented in Figure 3-1. Data on certain parameters that should not change with time and service need to be obtained only once. These appear on the flow diagram under the heading "Initial (Baseline) Evaluation Survey." Ideally, the tests for these parameters should be carried out as part of the acceptance testing of new concrete bridge components. While these characteristics will not change with time, their significance relative to the condition of the concrete does, and therefore the testing needs will vary depending on when they are carried out. Thus, air-void analyses (ASTM C457) would not be carried out on a structure over 5 years old because if the air-void system is not adequate to prevent freezing and thawing damage, it would already be evident in the condition of the structure. Likewise, most concrete materials, batching, or construction faults that would be revealed by petrographic examination (ASTM C856), such as retempering, sprinkling to aid finishing, and frost-susceptible aggregates will usually be manifested in the form of deterioration within five years. Alkali-carbonate reactivity, however, may take up to fifteen years to develop. Alkali-silica reactivity, which can be detected at very early stages by the new test recently developed under SHRP project C-202, may take up to 15 years to develop. However, it would not likely be evident in new (less than one year old) structures. Compressive strength, the primary indicator for concrete quality, is normally determined routinely by means of cylinder strengths (ASTM C39) or pullout tests (ASTM C900) at the time of construction. However, if not done at that time, compressive strengths of drilled cores (ASTM C42) should be done in conjunction with the baseline evaluation survey. Relative

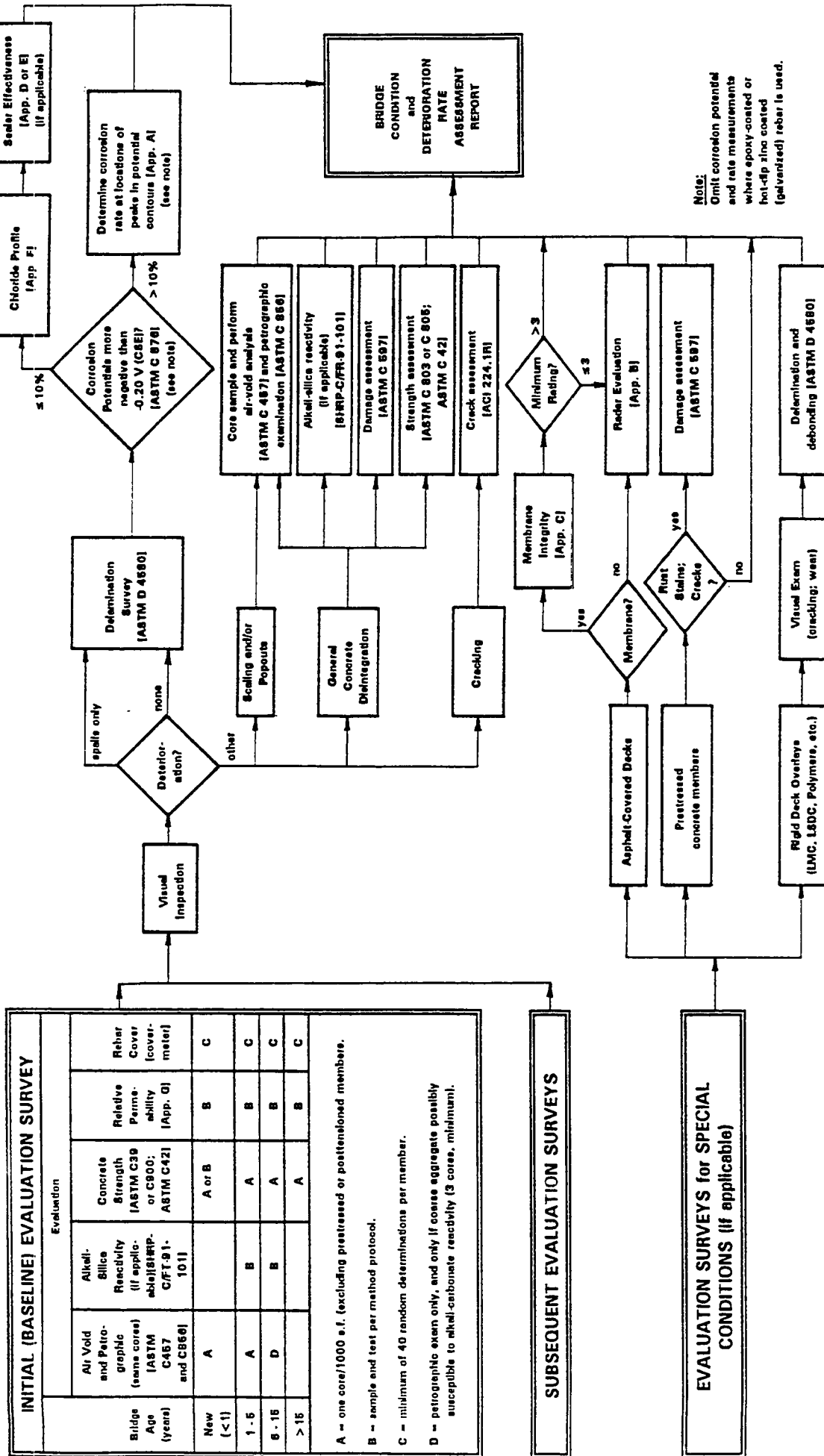


Figure 3-1. Flow diagram of the developed procedure for assessing the condition of concrete bridge components.

permeability testing, using the new technique described in Appendix G, is also a quality indicator test for the near surface concrete that may be done once as part of the baseline evaluation. Finally, a rebar cover survey using a commercially available covermeter is an essential test where chloride-induced corrosion of reinforcement may be a problem.

Subsequent Evaluations

Information on deterioration rates, as well as updates on condition, must be obtained by periodic subsequent evaluations. These are essential for the development and execution of rational bridge management policy. The initial step in the subsequent evaluations is a visual inspection for obvious signs of deterioration. Four types of deterioration may be observed:

- Spalling of the layer of concrete between the surface and the reinforcing bars;
- Scaling and/or popouts;
- Concrete disintegration; and
- Cracking.

A fifth form of deterioration, wear in wheel paths, is not included here because its cause and extent are immediately evident and require no further testing. However, it is an important matter relative to highway safety, which requires attention.

If there is no visible deterioration or only spalls, a delamination survey should be carried out. Delamination, the precursor of spalling, results from chloride-induced corrosion of the reinforcing steel. Sounding methods are recommended. Chain drags or electromechanical tapping devices are suitable for bridge decks (ASTM D4580), but tapping with a hammer or a steel rod is used for vertical and overhead surfaces.

Other testing for the ingress of chloride ions and resulting reinforcement corrosion should be carried out in concert with delamination testing. First, a corrosion potential survey should be conducted in accordance with ASTM C876. If 10 percent or less of the corrosion potential results are more negative than -0.20V relative to the copper-copper sulfate half-cell (CSE), chloride profiles should be established from samples taken at 0.5-in (13-mm) increments to the depth of the reinforcement using the new test method for chloride content detailed in Appendix F. The effectiveness of penetrating sealers (if used) should then be evaluated using either of the two new methods developed for this purpose and detailed in Appendices D and

E. If more than 90 percent of the corrosion potential results are more negative than -0.20V (CSE), an equipotential contour map of the concrete surface should be prepared as described in ASTM C876. At the locations of the peak negative values, as indicated by the equipotential contours, corrosion rates should be determined as described in the new method presented in Appendix A. Notice that corrosion potential and corrosion rate tests should not be carried out where epoxy-coated or hot-dip zinc-coated (galvanized) reinforcing steel is used. The matter of corrosion of epoxy-coated reinforcement is discussed in more detail later in this chapter.

If types of deterioration other than, or in addition to, spalling are observed during the visual examination, the emphasis on reinforcement corrosion is dropped. The rationale is that other types of deterioration are seated within the concrete mass, are progressive, and are probably not amenable to corrective action. Therefore, the corrosion problem, which can be combatted along several possible avenues, is irrelevant in the presence of the other progressive forms of deterioration. The other types are:

- scaling and/or popouts;
- general concrete disintegration; and
- cracking.

Scaling and/or popouts indicates freezing and thawing attack due to inadequately air-entrained concrete or frost-susceptible coarse aggregate particles, respectively. In such instances, air-void analyses (ASTM C457) and petrographic examinations (ASTM C856) on drilled core specimens should be carried out to pinpoint the cause and estimate the severity of the problem.

General concrete disintegration covers a host of concrete materials-related problems:

- freezing and thawing deterioration;
- alkali-silica reactive aggregates;
- alkali-carbonate reactive aggregates;
- sulfate attack;

- expansive oxidation/hydration reactions involving aggregates (e.g., pyrite/marcasite, periclase, gypsum, zeolites, clay minerals); and
- d-cracking (a freeze-thaw related mechanism usually associated with slabs on grade, but which might be found in bridge substructure elements).

There are several things that may need to be done to identify the problem and to define its extent in the case of general concrete disintegration. In terms of identifying the problem, air-void analyses (ASTM C457) and petrographic examinations (ASTM C856) should be carried out on drilled core specimens. If the situation warrants (i.e., possible presence of susceptible aggregates), alkali-silica reactivity testing in accordance with the new SHRP procedure should be done. As to the extent of damage, pulse velocity measurements, per ASTM C597, may be used. If strength assessment is deemed advisable, rapid, relative strength indications may be obtained with the rebound hammer (ASTM C805) or by penetration tests (ASTM C803). However, if a quantitative measure of strength is needed, compressive strength tests should be conducted on drilled core specimens (ASTM C42).

The cracking category applies to any form of cracking not covered by the preceding categories of deterioration:

- drying shrinkage cracking;
- plastic shrinkage cracking;
- stress-related cracking; and
- thermal cracking.

The evaluation of these forms of deterioration is beyond the scope of this procedure manual. For guidance in this matter, refer to the American Concrete Institute's *ACI Manual of Concrete Practice, Volume 3*, ACI 224.1R, "Causes, Evaluation, and Repair of Cracks in Concrete Structures."

Members Containing Epoxy-Coated Reinforcement

The development of epoxy-coated reinforcing steel at first indicated that corrosion of steel in concrete due to chloride penetration would no longer be a problem. However, this is not the case in all applications. There is extensive evidence that in severe chloride environments,

such as marine exposure, corrosion sometimes occurs. The epoxy coating in these instances appears to disbond from the steel at imperfections, but the exact mechanism of this disbondment is not known.

Since the possibility exists for corrosion in epoxy-coated steel, provisions should be made for routine inspection. The procedure should involve a visual inspection to locate any cracking, which may be associated with corrosion activity. It also should involve sounding of the concrete to locate any delaminations. These are the only two actions that are recommended, since any other tests performed might not provide information that is useful or conclusive.

Chloride ion profile tests are not suggested because the locations of the imperfections cannot be predicted and any profiles determined will most likely be in areas not likely to corrode. Potential surveys and corrosion rate measurement require connection to the reinforcement and electrical continuity among all reinforcement in the concrete. Since the epoxy coating acts as an electrical insulator, electrical continuity is prevented. In order to use potential surveys and corrosion rate devices, connection would have to be made to every reinforcing bar. This is not feasible due to high costs, and would provide numerous sites for the initiation of corrosion.

Evaluation Surveys for Special Conditions

Special evaluation surveys are needed for:

- asphalt-covered decks;
- pretensioned and post-tensioned prestressed concrete members; and
- rigid deck overlays.

Asphalt-covered decks are of two types--those where the asphalt overlay provides a traffic surface for a membrane deck protective system and those where the asphalt overlay is applied as the traffic surface on a bare concrete deck. The former instance usually involves an installation dating from the original deck construction or reconstruction and is intended to provide a barrier against chloride intrusion. Therefore, the primary concern here is the condition of the membrane. Membrane integrity should be evaluated using the new testing procedure outlined in Appendix C. If the test indicates the condition of the membrane to be suspect, the condition of the concrete deck should be evaluated using ground-penetrating, pulsed radar in accordance with the latest procedure described in Appendix B.

Asphalt overlays on bare decks usually are applied to provide acceptable traffic surfaces on deteriorated concrete decks. Since the asphaltic concrete will not prevent further deterioration (and, in the view of some, may even exacerbate problems by trapping deicing salt and moisture), such installations need to be monitored. This should be done using ground-penetrating, pulsed radar in accordance with the latest procedure described in Appendix B.

Condition assessment of prestressed concrete members is a critical issue. There are still no suitable, effective, non-invasive techniques to do this. For the present, the only alternative is to continue the current practice of visual inspection for rust stains and cracks. It is recommended that in the event of occurrence of these signs of distress, damage assessment be carried out using the pulse velocity techniques described in ASTM C597.

Bonded, rigid deck overlays consist of latex-modified concrete (LMC) low-slump dense concrete (LSDC), or polymer mortars. They may be part of the original deck construction (or reconstruction) or they may represent rehabilitation of deteriorated deck surfaces. In either case, the condition assessment should consist of a visual examination for cracking, spalling, and wear, and testing for delamination/debonding, using the methods described in ASTM D4580.

Tailoring Approaches for Specific Conditions

The approaches used by specific highway agencies, or even for specific bridges within the jurisdiction of given highway agencies, in applying the procedures outlined in this manual may be tailored to meet the unique conditions and needs encountered. Some jurisdictions or bridges will have no need for tests involving one or more of the following:

- **Materials or Environmental Considerations:**
 - alkali-silica or alkali-carbonate reactivity (no reactive aggregates);
 - air-void analysis (no freezing and thawing conditions); and
 - corrosion potential, corrosion rate, or chloride content (no chloride salt exposure, or epoxy-coated or galvanized reinforcement used).

- **Policy Considerations:**
 - sealer effectiveness (penetrating sealers not used);
 - membrane integrity (membranes not used);

- radar evaluation (decks not overlaid with asphaltic concrete); and
- condition of rigid deck overlays (rigid deck overlays not used).

With the diversity of climatic, environmental, material, and policy considerations that exist where this procedure manual may be put into practice, it should be evident to the user that at least some tailoring and discretion will be involved in most instances.

Documentation

The mode and extent of the documentation of test results is left to the discretion of the user of this procedure manual. Obviously, these considerations depend critically on the needs of the individual jurisdiction and requirements for data base compatibility with the individual agency's bridge management system. Ideally, an interactive video system using video laser discs for pictorial/graphics as well as numerical/descriptive data storage is suggested. Such systems are currently being developed in Denmark. A single laser disc can contain up to 50,000 high-resolution images or 300,000 pages of text that can be accessed in less than 0.5 of a second and viewed on a computer monitor. Minimally, paper records plus photos (or preferably VHS video) documenting chronological condition assessment data are required.

Appendix A

Standard Test Method for Determining Instantaneous Corrosion Rate of Uncoated Steel in Reinforced Concrete

Standard Test Method for

**DETERMINING INSTANTANEOUS CORROSION RATE OF UNCOATED STEEL IN
REINFORCED CONCRETE**

1. Scope

1.1 This test method covers the determination of instantaneous corrosion rate of uncoated steel in reinforced concrete.

1.2 The reinforced concrete discussed in the standard has, in general, a smooth (not scarred, cracked, or uneven) surface.

1.3 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

1.4 The concrete surface must be plain, with no liquid-impermeable overlays or coatings.

1.5 The depth of concrete cover over the uncoated steel is less than 4 in. (10.16 cm).

1.6 The reinforcing steel is not coated with epoxy, hot-dip zinc (galvanized), or any other coatings.

1.7 This method applies only to steel in direct contact with concrete, e.g., the ducts, but not the strands in ducted, posttensioned systems.

1.8 The reinforced concrete system is not cathodically protected by means of externally applied electric currents.

1.9 The reinforced concrete is not near areas of stray electric currents (e.g., power transmission lines) or strong magnetic fields.

1.10 The ambient temperature is greater than 40°F (5°C), and less than 100°F (38°C).

1.11 The concrete surface in the area of interest must be dry (no standing or flowing water, or visible moisture).

1.12 The values stated in English units are to be regarded as the standard. The values in parentheses are for information only.

2. Referenced Documents

2.1 *ASTM Standards*

A82 Standard Specification for Steel Wire, Plain, for Concrete Reinforcement

A496 Standard Specification for Steel Wire, Deformed, for Concrete
Reinforcement

C876 Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in
Concrete

D8 Definitions of Terms Relating to Materials for Roads and Pavements

G3 Standard Practice for Conventions Applicable to Electrochemical
Measurement in Corrosion Testing

3. Description of Terms

3.1 Reinforcing steel grid -- the layout of the reinforcing steel bars encased in concrete.

4. Summary of Test Method

4.1 This method employs the use of any one of three devices to electrochemically determine the instantaneous corrosion rate of uncoated steel reinforcing bars, plain (Specification A82) or deformed (Specification A496), in reinforced concrete. In general, the concrete surface should be in fair to good condition, exhibiting no cracks, scars, or large indentations. Also, the concrete surface may not have any coatings or overlays that are impermeable, or only minimally permeable, to water. The instantaneous corrosion rate is determined for a specific area of reinforcing steel, and the rate is expressed in current density units ($\text{mA} \cdot \text{ft}^{-2}$ [$\mu\text{A} \cdot \text{cm}^{-2}$]).

5. Significance and Use

5.1 Instantaneous corrosion rate of steel in reinforced concrete is needed to project the damage of concrete due to steel corrosion.

5.2 Successive measurement of instantaneous corrosion rates of uncoated steel in reinforced concrete at prescribed time intervals allows for estimation of service life.

6. Interferences

6.1 Stray currents, from causes such as power transmission lines or externally applied cathodic currents, may produce erratic results due to interference with each of the rate-determining instruments.

6.2 Water-impermeable coatings and overlays will interfere with results.

6.3 High electrical resistance of the concrete (as might result from unusually dry conditions, impregnation with dielectric materials, or "internally sealed" concrete) will interfere with obtaining accurate test results.

6.4 Readings should not be taken closer than about 12 in. (30 cm) from edges, joints, or other discontinuities that will disrupt current flow and will give erroneously high values for the polarization resistance.

7. Apparatus

7.1 The testing apparatus consists of the following:

7.1.1 A Device for Corrosion Rate Measurements -- Three devices are available as follows:

7.1.1.1 *KCC, INC. 3LP Device; Model 3LP-A (3LP device)* -- Manufactured by Kenneth C. Clear, Inc., USA. The device uses a large counter electrode (CE) (6.75 in. [17.1 cm] long) without current confinement. The operation is manual. Polarization is applied by continuous increase of cathodic current, and values of the current are recorded for

polarization values of 4, 8, and 12 mV. Only polarization resistance is determined (not the concrete resistance).

NOTE 1 – The KCC, Inc. 3LP Device is not patented nor proprietary. KCC, Inc. has offered loan of the instrument (as available) or plans and procedures free of charge to any state or federal agency. They will also allow these agencies to purchase the basic instrument from the company that builds it for them. For details, contact:

Mr. Kenneth C. Clear, President
KCC, Inc.
22455 Davis Drive, Suite 108
Sterling, VA 22170

7.1.1.2 *Portable Corrosion Monitor Model 3* -- Manufactured by the Nippon Steel Corporation, Japan (NSC device). The probe contains an Ag/AgCl reference electrode and a central CE (1.4-in.[3.6-cm] outer diameter), which is surrounded by a ring guard electrode (GE) (2.4 in. [6.0 cm] outer diameter). The device superimposes two current pulses of low and high frequencies, respectively, and enables the determination of both the polarization resistance and concrete resistance. The operation is automatic.

NOTE 2 – For details on the procurement and operation of the NSC Device, contact:

Mr. Kazumi Matsuoka
R&D Laboratories II
Nippon Steel Corporation
5-10-1 Fuchinobe Sagamihara
Kanagawa, 229 Japan

7.1.1.3 *The corrosion meter LG-ECM-03 and the probe LG-ECS-02 (GECOR device)* -- Manufactured by GEOCISA, Spain. The device uses a central CE (3.1 in [80 mm] in diameter) and an external ring GE, which confines the current to an area 5.5 in. (14

cm) in diameter. The equipment provides data on polarization resistance, corrosion current density (corrosion rate), and concrete resistance. The operation is automatic.

NOTE 3 – For details on the procurement and operation of the GECOR Device, contact:

Dr. J. Rodriguez
GEOCISA
Los Llanos de Jerez, 10y12
28820 COSLADA (Madrid)
Spain

7.1.1.4 All of these devices operate by measuring the polarization resistance, R_p . Polarization resistance is defined as the slope of the polarization curve at the origin. Corrosion current, I_c , is then calculated from the equation:

$$I_c = \frac{B}{R_p} \quad (\text{A-1})$$

where B equals 0.026 V when I_c is expressed in $\text{A}\cdot\text{cm}^{-2}$ and R_p is expressed in $\text{ohm}\cdot\text{cm}^2$.

7.1.2 *A half cell* -- The cell serves for determining half-cell potentials of steel rebars.

7.1.3 *Voltmeter* -- A voltmeter is used for measuring corrosion potentials. Its input resistance shall not be less than 10 $\text{M}\Omega$.

7.1.4 *Covermeter* -- A covermeter is used for locating rebars in concrete structures.

8. Materials and Reagents

8.1 *Sponges* -- Sponges are used to provide a low electrical resistance liquid junction between the concrete surface and the device for the corrosion rate measurements, and/or a half cell. The sponges for the NSC and GECOR device and for the half cell shall be about 0.2 in. (5 mm) thick. The sponge configuration must match the footprint of the probe.

8.2 *Electrical Contact Solutions* -- These solutions shall be used to wet the concrete surface and the sponges. They are as follows:

Potable water with wetting agent (95 ml of wetting agent (common liquid household detergent) mixed with 19 l of potable water) (ASTM C 876),

Potable water saturated with potassium chloride (for use with the NSC device), and potable water (for use with the GECOR device).

8.3 *Electrical Lead Wires*

8.4 *Tools* -- For establishing electrical connection with the reinforcing steel.

9. Sampling, Test Specimens, and Test Units

9.1 Select sites that have smooth concrete surfaces in fair or good condition, free from delaminations and large cracks or surface indentations, and that are subject to accurate reinforcing steel location determination.

10. Procedure

10.1 Visually inspect the reinforced concrete structure to determine which areas are to be tested. Criteria for area selection are left to the individual user.

10.2 Locate and mark the entire reinforcing steel grid in the area of interest with chalk or other water-permeable, nonconductive, nonmetallic marking material. Record the depth of cover and the steel bar diameters of all bars in the grid in the area of interest. Make note of any areas of steel bar overlaps or splices.

10.3 Make electrical connection with reinforcing steel grid in accordance with Test Method C876.

10.4 Determine the electrochemical potential in the region of interest, in accordance with Test Method C876, directly over the centroid of the reinforcing steel bar(s) of interest. Record this (these) potential(s). If a half-cell other than copper-copper sulfate (CSE) is used, convert the reading to CSE in accordance with Note 1 of ASTM C876.

10.5 Determine the corrosion rate using one of the three devices described at heading 7.1.

10.5.1 *Test Apparatus A: 3LP Device* -- Determine the surface area (SA) of the steel rebar under the probe from the relation: $SA = \pi D_{rb} L$, where D_{rb} is the diameter of the rebar, and L is the length of the polarizing electrode, equal to 6.75 in. (17.1 cm) for this device. Pre-wet the concrete surface of the chosen site and the probe sponge with potable water containing wetting agent. The wetted concrete area shall match the area of the probe. Weight the probe with a load of about 1 kg (2.2 lb). If the test surface is vertical or horizontal overhead, affix the probe with a rubber strap and screws set into the concrete.

Switch on the device and start the polarization procedure after the corrosion potential has almost stabilized (the potential drift should be less than about 2 mV/minute).

Polarize the object by manual continuous increase of cathodic current and record the current values for the polarizations of 4, 8, and 12 mV; the polarization should be completed in about 2 minutes. From the recorded change in potential (ΔE) and applied current (Δi), determine the polarization resistance (R_p): $R_p = \Delta E / \Delta i$.

10.5.2 *Test Method B: NSC Device* -- Determine the surface area of the steel rebar under the probe as for the 3LP device, with L equal to 1.2 in. (3.0 cm) (double width of the CE ring). Pre-wet the concrete surface of the chosen site with a saturated potassium chloride solution over an area matching the probe footprint (use a squirt bottle or a damp sponge). The chosen site shall be exactly over the centroid of the rebar. Wet the probe and a 0.2-in. (5-mm) thick sponge (diameter equal to that of the probe) thoroughly with the same potassium chloride solution, and place the sponge and the probe over the chosen site. Weight the probe with a load of about 0.5 kg (1.116). Attachment of the probe to vertical or overhead surfaces is achieved as indicated for the 3LP device. Switch on the device and record the readings of corrosion potential, polarization resistance, and concrete resistance. It is recommended that a high frequency pulse of 1,300 Hz and low frequency pulse of 0.02 Hz be used. In the event that the error indication (ERR) appears on the panel display, improve the electrical connection at the concrete/probe interface by additional wetting and loading. Also, set a lower current range on the instrument. The readings can be recorded after the

corrosion potential has almost stabilized (the potential drift should be slower than about 2 mV/minute). Usually, it takes at least 10 minutes to achieve stability of the potential.

10.5.3 *Test Apparatus C: GECOR Device* -- Determine the surface area of the steel rebar, with L equal to 5.5 in. (14.0 cm), and input this value (in cm) into the instrument. Pre-wet the concrete surface and the probe sponge with potable water. Weight the probe with a load of about 20 kg (44 lb). If the test surface is vertical or horizontal overhead, affix the probe with plastic straps and two screws set into the concrete. Wait until the corrosion potential is stable, then switch the device on and make the measurements of polarization resistance and concrete resistance. In case of erroneous indications, check the batteries and replace them, if required, and improve electrical contact at the concrete/probe interface by further wetting and additional loading of the probe.

10.5.4 Whatever instrument is used, make three measurements on each chosen site with the affixed probe. Be certain that the probe is centered over the rebars and the active corrosion sites (as indicated by the covermeter and half cell readings, respectively). Switch off the polarization between the measurements and disconnect the rebar, but do not detach the probe. The time interval between consecutive measurements shall be long enough to allow the half-cell potential to recover its initial value and stabilize. This requires at least 10 minutes.

11. Calculation and Interpretation of Results

11.1 *Calculation* -- Present corrosion rate as corrosion current I_c . Calculate I_c using the equation:

$$I_c \left[\frac{\text{A}}{\text{cm}^2} \right] = \frac{0.026 \text{ [V]}}{R_p \text{ [ohm} \cdot \text{cm}^2]} \quad (\text{A-2})$$

or

$$I_c \left[\frac{\mu\text{A}}{\text{cm}^2} \right] = \frac{26 \text{ [mV]}}{R_p \text{ [k ohm} \cdot \text{cm}^2]} \quad (\text{A-3})$$

where R_p is polarization resistance obtained from the measurements. For a rough evaluation it can be assumed that material loss is uniform and that a current density of $1 \mu\text{A} \cdot \text{cm}^{-2}$ corresponds to a steel loss of 0.0116 mm/year (0.433 mpy). Calculate mean value from the three measurements, and standard error of the mean (SEr) from the equation:

$$\text{SEr} = \frac{S}{\sqrt{n}} \quad (\text{A-4})$$

where S is standard deviation, and $n = 3$ (number of measurements).

11.2 Interpretation of Results

11.2.1 Laboratory testing of reinforced concrete specimens indicates the following regarding the corrosion occurrence:

$I_c < \text{about } 0.1 \mu\text{A}\cdot\text{cm}^{-2}$ indicates that the steel is in the passive state;

$I_c > \text{about } 1 \mu\text{A}\cdot\text{cm}^{-2}$ indicates that the steel is in the active or partially active state.¹

11.2.2 Values below $0.01 \mu\text{A}\cdot\text{cm}^{-2}$, if obtained, generally indicate a poor connection with the steel, the presence of a highly-resistive material in the overlay, or insufficient moisture, and should not be considered valid.

11.2.3 The 3LP, NSC, and GECOR devices give different values of I_c for the same site.² The GECOR device gives I_c values closest to the true values in laboratory tests.² However, there exist approximate linear correlations among the three devices, as given by the following equations.³

$$\log I_c \text{ (3LP)} = 0.47 + 0.84 \log I_c \text{ (NSC)} \quad (\text{A-5})$$

$$\log I_c \text{ (GECOR)} = -0.47 + 0.77 \log I_c \text{ (NSC)} \quad (\text{A-6})$$

$$\log I_c \text{ (GECOR)} = -0.90 + 0.92 \log I_c \text{ (3LP)} \quad (\text{A-7})$$

where I_c (3LP), I_c (NSC), and I_c (GECOR) are corrosion currents determined by the 3LP, NSC, and GECOR devices, respectively.

These equations show that the values obtained by the devices are interrelated and that any of the devices can be used for obtaining pertinent corrosion rate data.

12. Report

12.1 The report shall include the following information:

- 12.1.1 The type of device used.
- 12.1.2 The average ambient temperature.
- 12.1.3 A description of the measurement site (location, surface orientation, and visual inspection information).
- 12.1.4 The date of testing and the weather.
- 12.1.5 Values of corrosion currents obtained with the device, and of the half-cell potentials measured with a copper-copper sulfate (CSE) half cell or converted to CSE if determined with a different type of half cell. Also the concrete resistance should be included, if the device allows one to measure it.
- 12.1.6 Equipotential contour maps, showing the location of reinforcing steel contact, in accordance with Test Method C876, and the locations and values of the corrosion current readings taken.

13. Precision and Bias

13.1 *Precision* -- The single-operator standard deviation (1s) values have been found to be 3.716, 6.594, and 0.316 $\mu\text{A}/\text{in.}^2$ (0.576, 1.022, and 0.049 $\mu\text{A}/\text{cm}^2$) for Methods A, B, and C, respectively. Therefore, the results of two properly conducted tests by the same operator on the same material should not differ by more than 10.516, 18,658, and 0.890 $\mu\text{A}/\text{in.}^2$ (1.630, 2.892, and 0.138 $\mu\text{A}/\text{cm}^2$) for Methods A, B, and C, respectively (d2s).

13.2 *Bias* -- Since there is no accepted reference material suitable for determining the bias for the procedure in the test method for measuring instantaneous corrosion rate of uncoated steel in reinforced concrete, bias cannot be determined.

14. Keywords

14.1 Corrosion current, corrosion rate, electrochemical measurements, half-cell potentials, polarization resistance, reinforced concrete, reinforcing steel.

Endnotes

¹S. Feliu, J.A. Gonzalez, M.C. Andrade, and V. Feliu, "Determining Polarization Resistance in Reinforced Concrete Slabs." *Corros. Sci* 29, pp 105-113, 1989.

²J. Flis, S. A. Sabol, A. Sehgal, P. D. Cady, H. W. Pickering, and K. Osseo-Asare, "Electrochemical Measurements with Various Instruments on Steel Reinforced Concrete Bridges, Part I. Evaluation of Reinforcement Corrosion Rates." (Submitted to the National Association of Corrosion Engineers).

³ J. Flis, H. W. Pickering, and K. Osseo-Asare, Part II. Assessment of Applicability of Instruments for Corrosion Rate Evaluation." (Submitted to the National Association of Corrosion Engineers).

Appendix B

Standard Test Method for Assessing the Condition of Asphalt-Covered Bridge Decks Using Pulsed Radar

Standard Test Method for

**ASSESSING THE CONDITION OF ASPHALT-COVERED BRIDGE DECKS USING
PULSED RADAR**

1. Scope

1.1 This test method covers a procedure for noninvasive evaluation of the condition of concrete bridge decks overlaid with asphaltic concrete wearing surfaces using ground-penetrating, pulsed radar.

1.2 Specifically, the method predicts the presence or absence of delaminations (fracture planes) associated with the top or bottom reinforcing bar mats.

1.3 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

1.4 The values stated in English units are to be regarded as the standard. The values in parentheses are for information only.

2. Significance and Use

2.1 This test method provides information on the condition of concrete bridge decks overlaid with asphaltic concrete without necessitating removal of the overlay.

2.2 Information on the condition of the concrete deck is needed to estimate service life, to program rehabilitation and maintenance activities, and to estimate quantities for rehabilitation contracts.

3. Interferences

3.1 Abnormally shallow reinforcement cover may produce distortions that interfere with the detection of delaminations at the level of the top reinforcement mat, especially if the concrete deck is unusually dry.

3.2 Unusually wet conditions (visible surface moisture) may affect results.

3.3 Accumulations of soil, antiskid material, or other particulate debris in the curb areas may affect results, especially if damp.

4. Apparatus

4.1 The testing apparatus consists of the following:

4.1.1 Air-coupled, short pulse radar(s) with 1.0 ns monocycle pulse (6-in. [15.2 cm] free space resolution) and 50 scan/second data rate, minimum.

4.1.2 Data acquisition system, consisting of equipment for gathering radar data at the maximum data rate of radar system(s), 50 kHz for one radar, 100 kHz for two radars,

and 150 kHz for three radars. The system must be capable of accurately acquiring radar data with a 60 dB dynamic range.

4.1.3 Distance measurement system consisting of a marker wheel or equivalent with precision of ± 3 in. (± 7.6 cm).

4.1.4 All equipment must be installed on a vehicle with proper warning and safety equipment.

4.2 The apparatus used is illustrated in the functional block diagram appearing in Figure B-1.

NOTE 1 – Realtime digital data acquisition and high-speed radar signal processing equipment and software have been designed around the Penetradar Model PS-24 radar system to meet the needs of this test method. For additional details regarding the equipment, see the reference provided.¹ For details regarding availability, use, or licensing, contact:

Mr. Anthony J. Alongi
Penetradar Corporation
6865 Walmore Road
Niagra Falls, NY 14304.

5. Procedure

5.1 Radar inspection passes are to be made in a longitudinal direction and shall be parallel.

5.2 The lateral spacing between radar passes shall be a maximum of 3 ft (1 m) apart.

5.3 The longitudinal spacing between radar scans shall be a maximum of 6 in. (15 cm) apart.

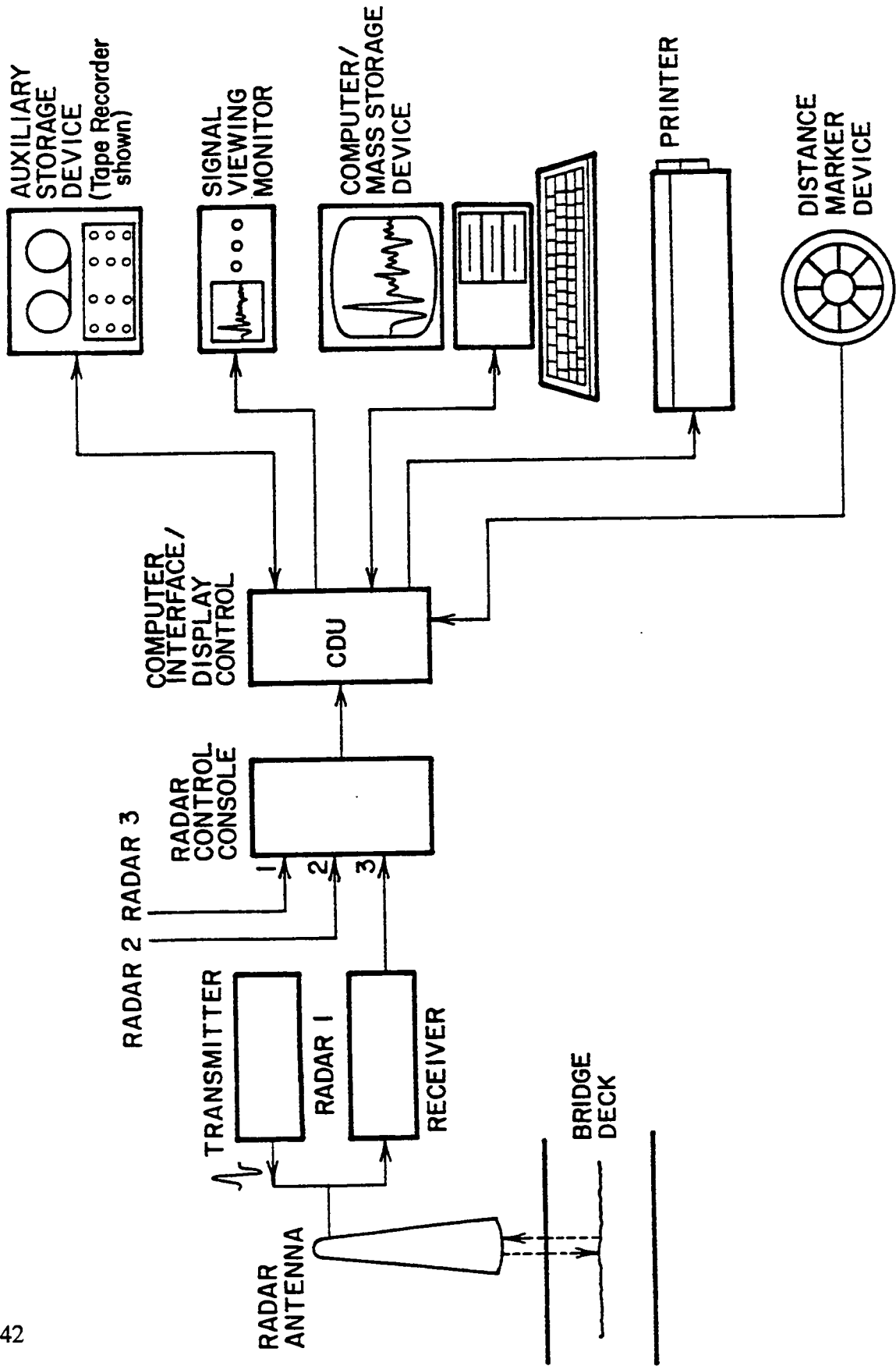


Figure B-1. Functional block diagram for one arrangement of the radar and support equipment.

5.4 Deck inspections should not be conducted if visible moisture is present on the bridge deck surface or in areas of accumulated soil, antiskid material, or other particulate debris.

6. Calculations

6.1 Delamination detection and quantity estimation for delaminations at the top reinforcing steel shall be conducted as follows:

6.1.1 Measure the signal strength, V_i , at the deck surface.

6.1.2 Determine the maximum signal strength of the deck bottom echo, V_{bs} ; for each longitudinal radar pass, $V_{bs} > = 0.0264 V_i$.

6.1.3 Measure the amplitude of the deck bottom echo, V_b , for each waveform.

6.1.4 If $V_b < = 0.385 V_{bs}$, then the concrete is delaminated.

6.1.5 Removal quantity = % delaminated \times deck area.

6.2 Delamination detection for delaminations at the bottom reinforcing steel shall be conducted as follows:

6.2.1 Calculate the mean value for the bottom rebar signal for each longitudinal radar pass, V_{avg} .

6.2.2 Measure the amplitude of the bottom rebar signal, V_{rb} .

6.2.3 If $V_{rb} > 1.5 V_{avg}$, then the concrete is delaminated.

7.1 Report

The report shall contain, as a minimum, the following:

- 7.1 Bridge identification and location,
- 7.2 Date and weather conditions,
- 7.3 General deck status relative to moisture and debris,
- 7.4 Any unusual conditions or circumstances, and
- 7.5 Radar results, in the following forms:

7.5.1 Percent delaminated per radar pass and for entire bridge deck in tabular form.

7.5.2 Plan view map of bridge deck, depicting radar inspection pass versus longitudinal distance and showing location and extent of detected delamination.

8. Precision and Bias

8.1 Insufficient data, to date, have been obtained for the determination of the precision and bias of this test method. However, for a sample of 10 bridge decks in New York, Virginia, and Vermont, an average error in radar prediction of $\pm 11.2\%$ occurred with respect to top reinforcement delaminated area as determined from ground truth (chain drag, core samples, and actual repair quantities).

9. Keywords

9.1 Asphalt-covered decks, bridge decks, delaminations, ground-penetrating radar, nondestructive testing, radar.

Endnote

¹"Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion," Volume 3: "Method For Evaluating the Condition of Asphalt-Covered Decks," Strategic Highway Research Program, National Academy of Sciences.

Appendix C

Standard Test Method for Determining the Condition of Preformed Membranes on Concrete Bridge Decks Using Pulse Velocity

Standard Test Method for

**DETERMINING THE CONDITION OF PREFORMED MEMBRANES ON CONCRETE
BRIDGE DECKS USING PULSE VELOCITY**

1. Scope

1.1 This test method covers the determination of the degree of deterioration of membrane systems installed on reinforced concrete bridge decks that have been overlaid with hot-mix asphalt.

1.2 The deterioration of membrane systems on bridge decks and their debonding from the hot-mix asphalt overlay and/or the concrete substrate is determined by effects on the transit time of ultrasonic pulses.

1.3 The values stated in SI units are to be regarded as the standard.

1.4 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. Referenced Documents

2.1 ASTM Standards:

C 597-83 Standard Test Method for Pulse Velocity through Concrete

3. Summary of Test Method

3.1 An ultrasonic pulse is transmitted into the bridge deck by a piezoelectric sending transducer placed on the bridge deck surface. A portion of the energy is reflected back to the surface by discontinuities encountered by the wave front during its passage through the deck. The depth and nature of the discontinuity will alter the path and the velocity of the reflected pulse.

3.2 A second transducer placed on the surface of the bridge deck at a distance of 89 mm (3.5 in.), center to center, from the sending transducer receives the reflected wave front. Through the use of timing circuits, the elapsed time for the earliest part of the pulse transmitted and received is displayed in microseconds (μsec).

4. Significance and Use

4.1 This test may be used to assess the integrity of membrane systems used in bridge decks as chloride barriers, to indicate any debonding between the membrane and the bridge deck or the asphalt overlay, perforations or deterioration of the membrane, or a combination of these.

4.2 The transit time of the ultrasonic pulse is used to estimate the relative condition of the membrane system in terms of a condition rating scheme.

5. Interference

5.1 This test shall not be used on a cracked asphaltic surface. A rough pavement surface shall be carefully covered with a thin layer of petroleum jelly in order to provide an even coupling surface, without surface voids, for the transducers.

6. Apparatus

6.1 The testing apparatus¹ consists of a pulse generator, a transmittal transducer and a receiver transducer, a time measuring circuit, a time display unit, connecting cables, and a time trigger.

6.1.1 *The Pulse Generator* -- The pulse generator should have a range of resonant frequency of 20 to 500 KHz, should be housed in a protective case, and operable on AC or DC current.

6.1.2 *Transducers* -- The transducers shall be piezoelectric transducers operating at 54 KHz using a suitable couplant material (petroleum jelly).

6.1.3 *The Connecting Cables* -- The co-axial connecting cables shall be 3 m (10 ft) long to achieve an acceptable resolution.

6.1.4 *The Timing Device* -- The timing device shall be accurate to $\pm 0.1 \mu\text{sec}$, and shall not be affected by temperature, humidity, or voltage change. The display unit shall have a sensitivity of $0.1 \mu\text{sec}$.

6.1.5 *Calibration Device* -- A calibration device shall be provided with the instrument to check the operation of the time-measuring circuit.

7. Procedure

7.1 *Timing Device Calibration* -- Calibration of the timing device shall be carried out when the instrument is turned on and before any measurements are taken. The timing device is calibrated according to the known transit time of the calibration device.

7.1.1 Wait 20 to 30 minutes and repeat the calibration to ensure repeatability.

7.1.2 Repeat the calibration at least every 4 hours during testing periods.

7.3 Transducers are positioned using a wood template with two holes having the same diameter as the transducers and spaced 89 mm (3.5 in.) apart, center to center. The transducers' surfaces are covered with a petroleum jelly layer, which acts as a couplant.

7.4 The transducers are placed on the pavement surface tightly and carefully. Then the transducers are twisted, 10 to 25 degrees, under hand pressure to ensure sufficient coupling and interlocking with pavement surface.

7.5 The timing trigger is activated and the transit time is recorded. No external pressure is applied on the transducer when the timing trigger is activated.

7.6 Transducers and the wood template are cleaned and prepared for the next measurement.

8. Calculations

8.1 Membrane system condition ratings are based on an arbitrary numerical scale of 0 to 10; 0 indicates a deteriorated membrane with no bonding to either the asphaltic layer or to the concrete substrate, while 10 indicates a sound membrane with a good bonding to both the

asphaltic overlay and to the concrete surface. A rating below 3 indicates an ineffective membrane.

8.2 The membrane rating is calculated from the following equation:

$$\text{MemRate} = -7.39 + 0.668 \text{ Vmeter} \quad (\text{C-1})$$

where

MemRate = membrane rating; and

Vmeter = pulse transit time in microseconds.

9. Report

9.1 The report shall include the following:

9.1.1 *The transit time.*

9.1.2 *The membrane rating.*

9.1.3 *The location of transducers.*

9.1.4 *Identification of the test site.*

10. Precision and Bias

10.1 *Precision* -- The single-operator, within-laboratory standard deviation has been found to be $0.15 \mu\text{s}$ (1s). Therefore, results of two properly conducted tests by the same operator on the same material should not differ by more than $0.4 \mu\text{sec}$ (d2s).

10.2 *Bias* -- The procedure in this test method for evaluating the condition of preformed bridge deck membranes, in situ, based on ultrasonic pulse transit time has no bias because the value of transit time (or pulse velocity) is defined only in terms of this test.

Endnote

¹A commercial device suitable for meeting the needs described here is available in the "V-Meter," which is distributed by most suppliers of concrete testing equipment including Hogentogler and Co., Inc., Humboldt Mfg. Co., James Instruments, Inc., Rainhart Co., Soiltest, Inc., and Triggs Technologies, Inc.

Appendix D

Standard Test Method for Determining the Relative Effectiveness of Penetrating Concrete Sealers by an Electrical Resistance Method

Standard Test Method for

DETERMINING THE RELATIVE EFFECTIVENESS OF PENETRATING CONCRETE SEALERS BY AN ELECTRICAL RESISTANCE METHOD

1. Scope

1.1 This test method covers the determination of the changing electrical resistance between two stripes of conductive paint applied to a concrete surface. The test method can be used as an indication of the relative effectiveness of penetrating sealers applied to concrete, but does not determine the actual resistivity of the concrete.

1.2 The surface resistance measured by this test method is independent of the dimensions of the concrete provided that at least 1 in. (25 mm) of clearance is allowed to the nearest edge of the concrete under test.

1.3 The method is applicable to both laboratory specimens and field structures over a temperature range of 50°F (10°C) to 120°F (49°C).

1.4 The values stated in inch-pound units (or cgs units) are to be regarded as the standard where inch-pound units (or cgs units) are given first, followed by SI units. Where

only SI units are given, or where SI units are given followed by inch-pound units (or cgs units), the SI units are to be regarded as standard.

1.5 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use. Specifically the paint and the propane used in this test method contain materials which, in certain concentrations, may either be flammable or require special handling. Material safety data sheets from OSHA, DOT, and ACGIH should be consulted for appropriate precautions. The services of a Certified Safety Professional (CSP) or a Certified Industrial Hygienist (CIH) will assist in establishing prudent practice.

2. Summary of Test Method

2.1 The method is based on measurement of the 100-Hz electrical resistance between two narrowly spaced stripes of conductive paint applied to a concrete surface. The paint curing is accelerated, and the concrete surface is preconditioned by heating to approximately 120°F (49°C) using either a small infrared propane heater or an electrical blow dryer. The test area is wetted for a short period of time. Excess surface water is removed and a resistance measurement after 4 minutes is an indication of the ability of penetrating surface sealers to prevent water penetration or to expel the water from the surface layer. The absolute resistance across the fixed geometry gage is a qualitative measure of the

effectiveness of the sealer. Combined with a measurement on uncoated sections of the same concrete, the measurements become semi-quantitative, and the effectiveness of the sealers can be classified as ineffective, borderline effective, or effective.

3. Significance and Use

3.1 This method can be used to gain an indication of the relative effectiveness of penetrating sealers applied to concrete. It can also be used as a research tool to compare uncoated concretes as to their surface conductivity caused by variations in soluble salts, alkalies, and porosity.

3.2 This method is applicable in both field and laboratory provided that the test surface is almost flat, without major cracks, and that the center of the test area is no closer than approximately 1-in. (25 mm) from the edge of the member or specimen.

4. Interferences

4.1 The test can be sensitive to surface roughness. Tests cannot be carried out on grooved or tyned (skid-textured) surfaces. Very rough or weathered surfaces may also pose problems. If the end-to-end resistance of the individual sides of the resistance gage cannot be reduced below approximately 125 ohms (DC measurement), even on repeat installation of the gage, roughness, bug-holes, or cracks in the concrete may be the reason.

5. Apparatus and Supplies

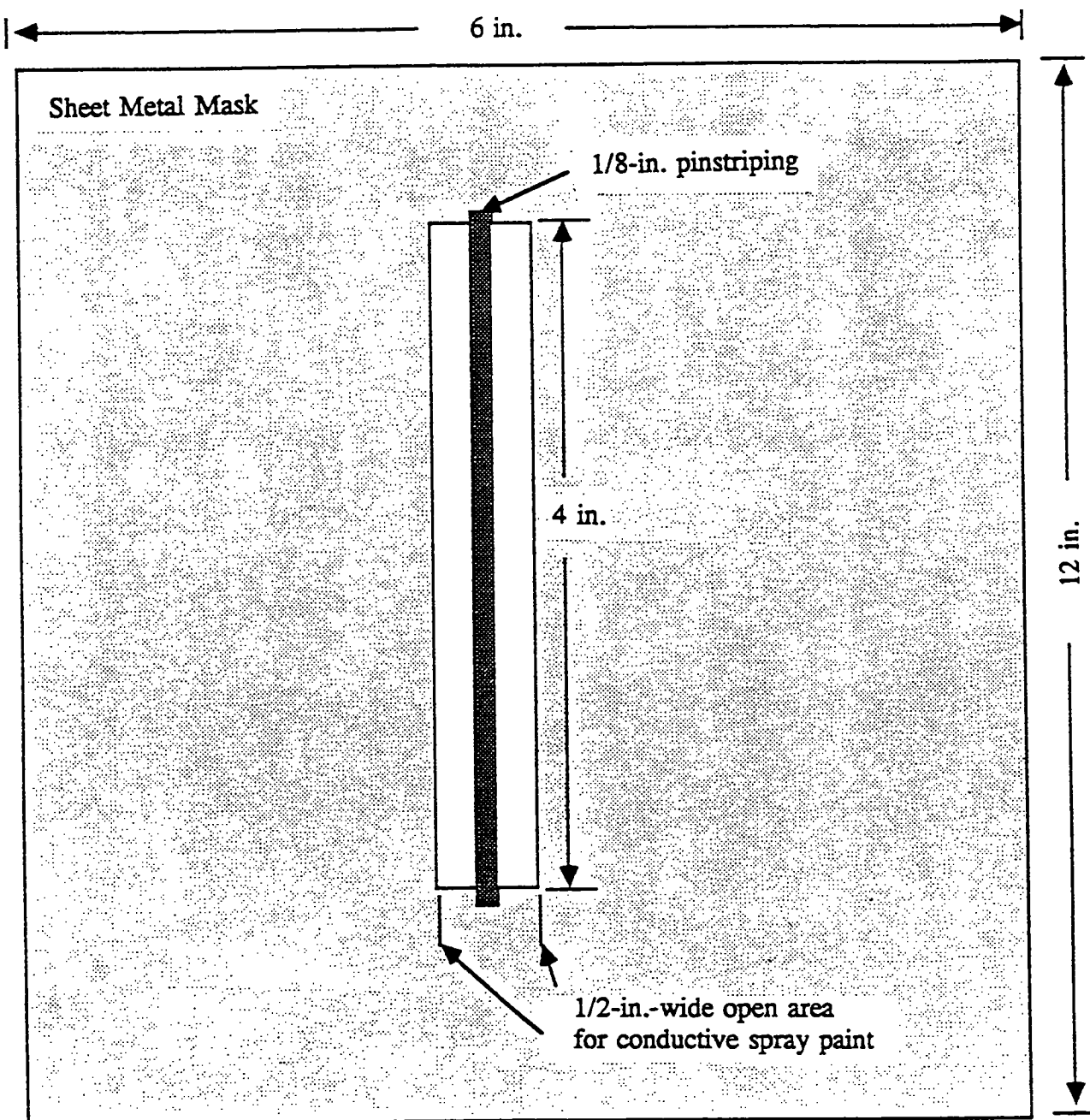
5.1 The required equipment consists of a user-prepared painting mask, conductive spray paint, a small propane infrared heater or an electrical blow dryer, a digital readout temperature indicator with flexible thermocouple, a standard digital multi-meter, a 100-Hz AC ohm meter, and spring-loaded test leads for one-handed operation.

5.1.1 *Mask* -- The mask for applying the two paint stripes to the concrete is shown in Figure D-1. The overall size of the mask is not critical as long as it can catch some of the over-spray. In use, the mask is centered over a strip of Scotch 3M 218 Fine Line Tape or equivalent 1/8 in. (3 mm) wide. The mask is held to vertical surfaces with a good grade of duct tape 3 in. (75 mm) wide. On horizontal surfaces, a suitable small weight can be used to hold the mask in position.

5.1.2 *Paint* -- A paint suitable for use with this test method is E-KOTE- 40 Silver Conductive Paint or equivalent.

5.1.3 *Heater* -- A suitable portable infrared heater is the Magna-252 Infrared Tool & Heater or equivalent. This heater operates from a standard 14.1-oz (415-ml) propane fuel cylinder. An alternate heater would be either an AC- or DC-operated hand-held blow dryer.

5.1.4 *Temperature Indicator* -- Any suitable thermocouple digital thermometer having a resolution of 1°F (0.5°C) with a flexible, thin wire copper-constantan thermocouple



Note: 1 in. = 2.54 cm

Figure D-1. Mask for production of surface electrodes (not to scale).

is acceptable. Liquid crystal temperature-indicating labels also can be used, but with less resolution.

5.1.5 *Digital Multi-Meter* -- A digital multi-meter having a DC resistance range of 0 to at least 20 megohms and input impedance of at least 10 megohms.

5.1.6 *AC Resistance Meter* -- A suitable 100-Hz AC resistance meter is the Model 400 Solid State 4-Pin Soil Resistance Meter from Nilsson Electrical Laboratory, Inc., New York, NY 10011. The highest measurable resistance of this unit is 1.1 megohms.

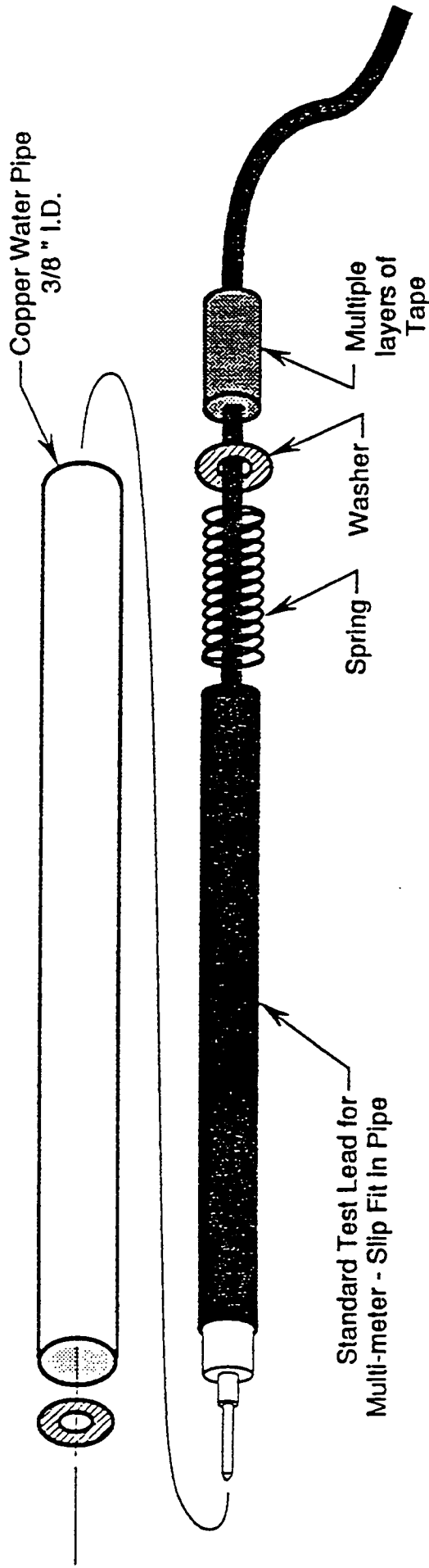
5.1.7 *Test Lead* -- Spring-loaded test leads for measuring the electrical resistance of the gage as shown in Figure D-2.

6. Sampling

6.1 *Spacing between Measurements* -- The spacing between measurements should be consistent with the size of the member being investigated and the intended use of the measurements.

7. Procedure

7.1 *Surface Preparation* -- The surface must be clean and free from grooves, cracks, and irregularities, which could prevent obtaining a good gage application. The surface must be dry to the touch for the paint gage to bond properly. If the surface is wet or the ambient



Assembly:

1. Insert test lead into pipe.
2. Solder washer to pipe.
3. Tape will act as stop.
4. Tape two assembled pipes together to form a one-handed dual prong test lead.

Note: 1 in. = 2.54 cm

Figure D-2. One-handed dual prong test lead.

temperature is near 50°F (10°C) heating to 120°F (49°C) will facilitate gage application. The surface should then be brushed and gently scrubbed with a dry paper towel to remove dust, dirt, or debris prior to testing.

7.2 Attaching the Mask -- A 6-in. (152-mm) strip of the fine line tape is applied to the area to be tested. The tape is pressed onto the concrete by applying heavy thumb pressure from the center and working to the ends of the tape. The metal mask is then centered over the tape. On vertical surfaces, the mask is held in place by four strips of duct tape 3 in. (75 mm) wide. On horizontal surfaces, a small weight can be used to hold the mask in place. The tip of the copper-constantan thermocouple is taped securely with thin transparent tape to the metal mask, next to the slit.

7.3 Gage Application -- The normal cure time of the paint is 16 hours. To accelerate the cure, the following procedure is used. Shake the paint can thoroughly per manufacturer's instructions. Hold can approximately 10 in. (250 mm), or somewhat closer in windy conditions, from the mask surface. Direct the spray near the end of the metal slit. When the paint flow has stabilized, pass the spray lengthwise over the slit six times, alternating the direction at the end of each sweep. The coats should be thin enough to prevent any runoff or seepage under the fine line tape. Heat the surface with either the infrared heater or a blow dryer for 5 minutes. Control the indicated temperature at 120°F (49°C). Repeat the gage application and the heating cycle two additional times. Remove the mask and the fine line tape.

NOTE 1 – Attempts to make a non-conductive gage acceptable by curing at a higher temperature, even as low as 135°F (57°C), are not recommended. This may cause changes in the concrete or the sealer, making a comparison with normal cure gages invalid. Occasionally, a high-resistance gage can be made conductive by installing one more layer of paint followed by curing at the standard 120°F (49°C).

NOTE 2 – Occasionally, the quality of a paint may be the reason for a high end-to-end resistance of the gage. Switching to a new can or batch of paint may be necessary. With the paint used for the development of the test it was generally found that a total of eight gages could be made reliably from each can. If the sealer, especially of the epoxy type, has not cured adequately either because of very recent application or low ambient temperature, it may be impossible to prepare a low-resistance gage. This is presumed to be due to remaining solvents or resins bleeding into the paint, preventing a low-resistance path between the conductive pigment particles in the paint.

7.4 *Preliminary Testing of Gage* -- Measure the end-to-end DC resistance of the two sides of the gage. Resistances of up to approximately 125 ohms are acceptable. A very good gage will be in the range of 5 to 15 ohms. Record the readings. Measure the DC insulation resistance between the two sides of the gage. Record the reading in megohms. A dry concrete containing low amounts of soluble salts will have a resistance exceeding 20 megohms. Gages having a resistance exceeding 5 megohms can be used (see note 3).

NOTE 3 – The DC insulation resistance (side-to-side) of a gage normally is greater than 20 megohms. Side-to-side DC resistances as low as 2.5 megohms can be acceptable where an approximate error of 20% in the actual 100 Hz resistance measurement will not affect the conclusion as to the acceptability of a particular sealer. Low insulation resistances may be caused by excess levels of alkalis or soluble salts in or on the concrete. It may be possible to wash off a sufficient amount to produce gages of adequate insulation resistance.

7.5 AC Resistance at 100 Hz -- Wet the gage with potable water and keep wetted for 5 minutes. Immediately dry the gage by pressing a dry, folded paper towel against the gage for 5 seconds. Follow this by gently wiping the gage in a lengthwise direction using a crumpled, dry paper towel. Take the AC resistance reading 4 minutes after wiping the gage.

NOTE 4 -- A vertical area can be kept wetted by holding a wet sponge over the gage and pouring water on the top of the sponge.

NOTE 5 -- The Nilsson meter can be held temporarily in the active low-sensitivity measurement mode by attaching a rubber band between the toggle switch and the left locking mechanism.

NOTE 6 -- Optional, additional information can be obtained by taking resistance readings at 0, 1, 2, 3, 4, and 10 min.

8. Interpretation of Results

8.1 Use Table D-1 to interpret the results. These values were selected as representative from gage measurements both on field and laboratory concretes.

Table D-1. Categories of relative effectiveness of sealers.

100 Hz Resistance (kiloohms)	Relative Effectiveness of Sealer (Category)
0 to 200	Ineffective (or not sealed)
200 to 400	Borderline effective
over 400	Effective

9. Report

9.1 The report shall consist of at least the following:

9.1.1 Location of test or identification of specimen,

9.1.2 Specimen history or recent environmental field conditions,

9.1.3 Conditions at time of test (air temperature, concrete temperature, RH, wind speed, etc.),

9.1.4 Pre-conditioning of test area (if used),

9.1.5 Preliminary gage test results,

9.1.6 AC resistance at 4 min and category, and

9.1.7 Optional resistance measurements at other time intervals.

10. Precision and Bias

10.1 *Precision* -- The single-operator within-lab coefficient of variation has been found to be 19% of the resistance reading. Therefore, results of two properly conducted tests by the same operator on the same material should not differ by more than 54%.

10.2 *Bias* -- The procedure in this test method for measuring relative effectiveness of penetrating concrete sealers by electrical resistance has no bias because the surface resistance is defined only in terms of this test method.

11. Keywords

11.1 Bridges, concrete, electrical resistance, in situ testing, nondestructive testing, penetrating sealers.

Appendix E

Standard Test Method for Evaluating Penetrating Concrete Sealers by Water Absorption

Standard Test Method for

EVALUATING PENETRATING CONCRETE SEALERS BY WATER ABSORPTION

1. Scope

1.1 This test method is designed to evaluate the effectiveness of penetrating sealers applied on concrete surfaces from observation of the surface absorption of concrete.

1.2 This method gives a rating for the effectiveness of sealers (good or poor) following criteria developed based on a laboratory investigation.

1.3 The method is applicable to both laboratory specimens and field structures.

1.4 The values stated in inch-pound units (or cgs units) are to be regarded as the standard where inch-pound units (or cgs units) are given first, followed by SI units. Where only SI units are given, or where SI units are given followed by inch-pound units (or cgs units), the SI units are to be regarded as standard.

1.5 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. Summary of Test Method

2.1 This method is based on the measurement of surface absorption of water under low pressure. A water reservoir (cell) with a capillary tube threaded into the top end is affixed to the concrete surface. Modeling clay into which grease has been kneaded is used to affix and seal the cell to the surface to be tested. Water is then allowed to flow from an external reservoir into the cell until the water column is filled to a height of approximately 40 cm (15.7 in.). The intake tube is closed, and the water column drop is monitored as a function of time over 10 minutes. Effectiveness of a sealer is then expressed in a qualitative manner (good or poor) based on criteria developed in the laboratory.

3. Significance and Use

3.1 This method gives an indication of the ability of a penetrating sealer to prevent the intrusion of water or salt-laden water into concrete.

3.2 This method is applicable for both vertical and horizontal concrete surfaces, but the test surface has to be flat and not grooved or tyned.

3.3 This method is intended to be rapid and nondestructive for the convenience of field applications, especially on in-service bridges and highway structures where tests are desired to be conducted within a short period of time.

4. Interferences

4.1 Moisture content of the concrete can affect the test results, and drying may be required in order to obtain more reproducible results. In some cases where the concrete has been exposed to rainwater for a significant period of time prior to performing the test, drying can be accomplished by subjecting the surface to drying for a period of 10 minutes at 120°F (48°C).

NOTE 1 – If the concrete surface is only temporarily exposed to moisture (such as during a brief summer shower) and allowed to dry under warm natural conditions for 2 hours or more, artificial drying may not be required.

4.2 In field applications, air temperature can influence the test results. If the test has to be performed in severe weather, either hot or cold, the concrete surface should be brought to a relatively moderate temperature (60°F to 80°F, 15°C to 27°C) by either cooling or heating the concrete surface. However, it is more practical simply to schedule tests during moderate weather conditions.

4.3 The test can be sensitive to surface roughness. Tests cannot be carried out on grooved or tyned (skid-textured) surfaces.

4.4 The test can be sensitive to the quality of concrete. The same sealer might give different results if tested on different concretes. Better results will be obtained on high-quality concrete. However, in laboratory evaluation it is recommended that a control specimen (an unsealed specimen made of the same concrete tested under the same conditions)

be used to obtain a more appropriate evaluation. In field applications, if sections that were not sealed can be located on the same structure, then these sections may also serve as control areas for field tests.

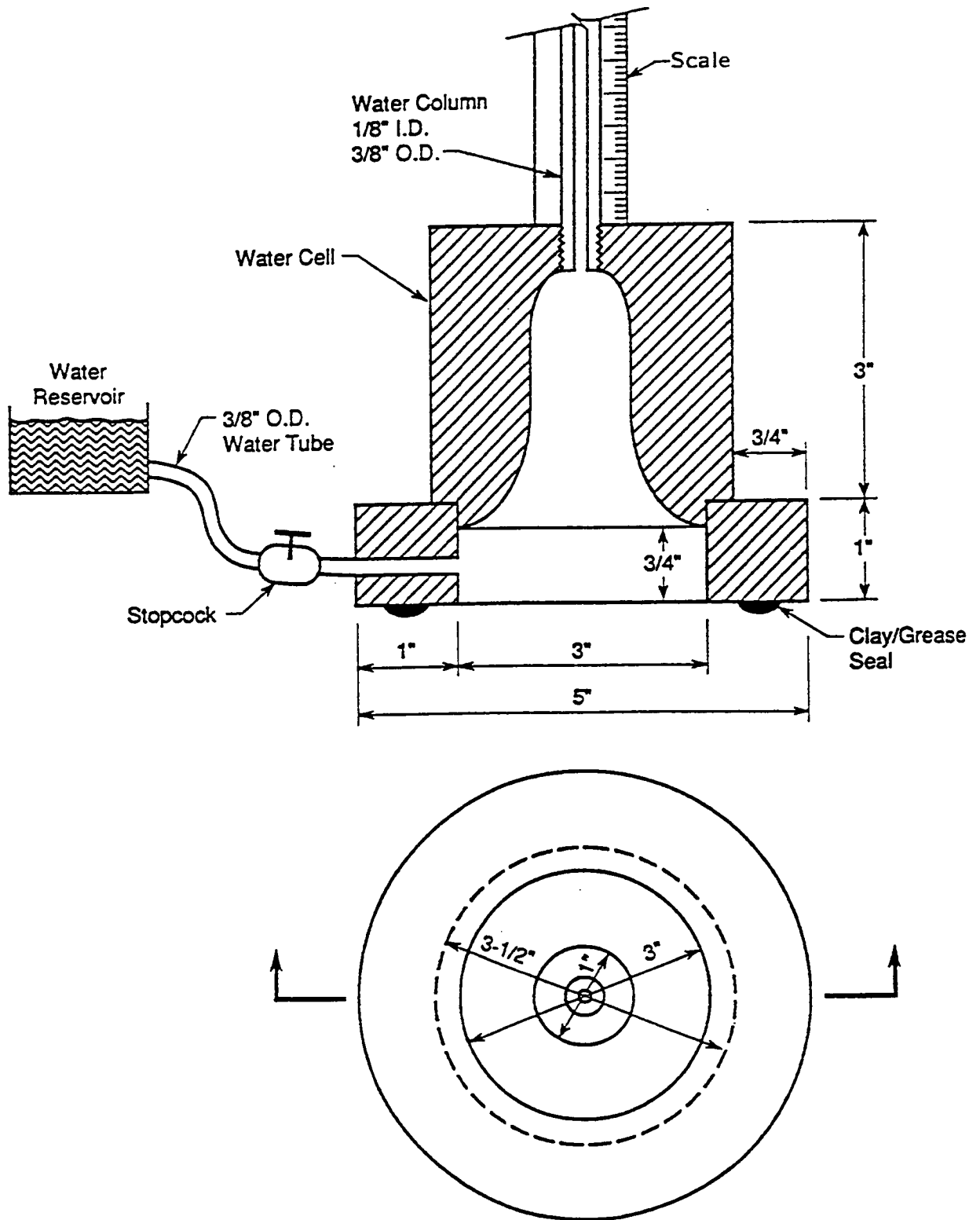
5. Apparatus

5.1 The testing apparatus for horizontal surfaces (Type A) is shown in Figure E-1 and the testing apparatus for vertical surfaces (Type B) is shown in Figure E-2. They consist of water cell, capillary tube and scale, water reservoir, flexible tube (intake tube), and flow control device (stopcock).

NOTE 2 -- The apparatus for vertical surfaces (Type B) is, in principle, the same as Type A except that the configuration of the cell has been modified to be used on vertical surfaces.

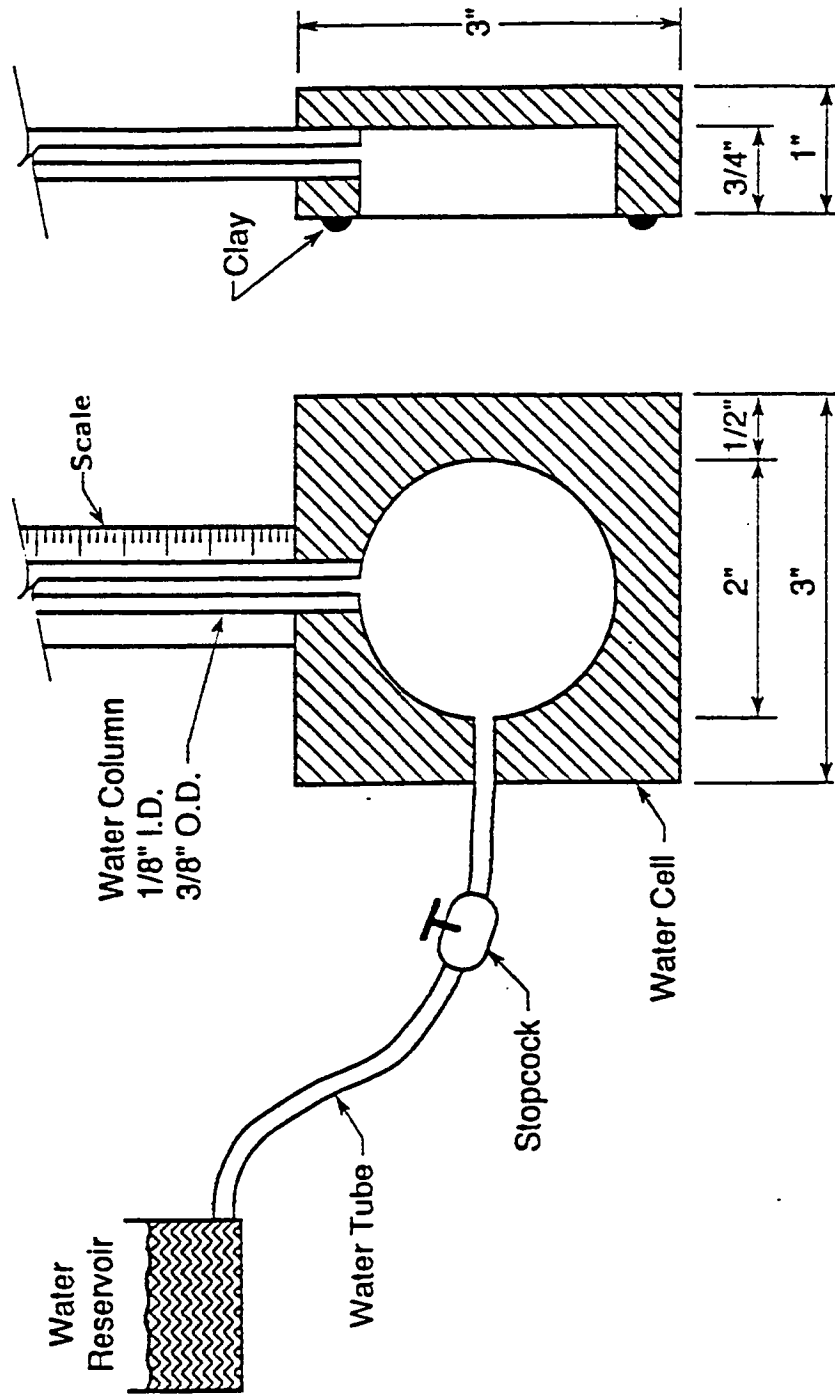
5.1.1 *Water Cell* -- The water cell shall be made of suitable impermeable material (clear acrylic is preferred). Two holes should be drilled into the cell, one for the water column and one for the intake tube.

5.1.1.1 *Water Cell Type A (Horizontal Surface)* -- As shown in Figure E-1, the cell should have a 3-in. (75-mm) diameter contact area with the concrete surface and a 1-in. (25.4-mm) wide edge to provide space for sealing the cell to the surface with a modeling clay/grease mixture. A 0.375-in. (9.5-mm) hole is drilled into the top of the cell for the capillary tube (water column) and another hole of the same size is drilled into the edge for the plastic tube connected to the stopcock.



Note: 1 in. = 2.54 cm

E-1. Modified RILEM II.4 device for use on horizontal surfaces.



Note: 1 in. = 2.54 cm

E-2. Modified RILEM II.4 test piece for use on vertical surface.

5.1.1.2 *Water Cell Type B (Vertical Surface)* -- As shown in Figure E-2, this cell should have a 2-in. (50.8-mm) diameter contact area. The outer side of the contact area is reserved for the sealing material (modeling clay). A 0.375-in. (75-mm) hole is drilled on the top edge for the capillary tube, and another hole is drilled on the side edge for the tube connected to the stopcock.

5.1.2 *Water Reservoir and Flexible Intake Tube* -- The water reservoir shall have a capacity of 1 L of water. The flexible tube (preferably plastic) with an outer diameter of 0.375 in. (9.5 mm) and inner diameter of 0.250 in. (6.35 mm) is connected to the water reservoir from one side and to the stopcock from the other side. The stopcock size should be chosen to fit into the flexible tube.

5.1.3 *Capillary Tube and Scale* -- The capillary tube (water column) shall be made of clear material (acrylic plastic) with a 0.125-in. (3-mm) inner diameter and 0.375-in. (9.5-mm) outer diameter, with a length of 19.6 in. (500 mm). A wooden or plastic scale with a minimum length of 19.6 in. (500 mm) graduated in millimeters is attached to the capillary tube. The zero reading starts from the top of the water cell.

5.1.4 *Balance* -- A balance accurate to within 0.1% of the weight of sample.

5.1.5 *Stopwatch* -- A stopwatch or clock capable of being read to an accuracy of 1.0 seconds.

6. Sampling

6.1 *Spacing between Measurements* -- The spacing between measurements should be consistent with the size of the member on which the sealer is applied.

7. Procedure

7.1 *Clay Preparation* -- Add 5 g (0.01 lb) of grease to 50 g (0.11 lb) of clay, and mix them by hand until all of the grease is dispersed into the clay and a homogeneous mix is achieved. Shape the mix into a ball and roll it between two flat surfaces until a uniform cylindrical bar with a length of 12 to 13 in. (305 to 330 mm) is obtained. The length of the clay bar required for the Type A cell is about 12 in. (305 mm) and for the Type B cell, about 10 in. (250 mm).

NOTE 3 -- Commercially available multi-purpose grease can be used for this purpose. Oil-based modeling clay can be used; a gray color is preferred, to match the tone of the concrete surface.

NOTE 4 -- A nonuniform rolled clay bar may lead to water leakage between the cell and concrete surface.

7.2 *Surface preparation* -- The surface must be dry, clean, and free from grooves, cracks, and irregularities, which could cause water leakage from the cell. If significant rainfall has occurred during the past 24 hours and the surface is now in a surface-dry condition, it may be necessary to spot-dry each test location selected. This can be accomplished by subjecting the surface to drying for 5 to 10 minutes at a temperature of 120°F to 130°F (48°C to 54°C).

7.3 Determination of Water Column Drop -- Place the clay bar around the edge of the water cell and put the cell on the pre-located testing spot. Press the cell strongly to the surface until the clay is spread underneath the edge of the cell. Close the stopcock and fill the water reservoir with clean water. Open the stopcock and let the water flow into the cell and capillary tube until the capillary tube is filled to a height of approximately 40 cm (15.75 in.). The height of the water column can be controlled by moving the reservoir up and down. Close the stopcock, start the timer, and take the first reading. Take a reading every minute up to 10 minutes, then stop the test by opening the stopcock and draining the water back to the water reservoir.

NOTE 5 -- To apply uniform pressure when pressing the cell to the surface, a 5- x 10- x 1-in. (125- x 250- x 25-mm) plywood piece with a 1-in. (25-mm) hole in the center can be used to apply pressure on the horizontal cell using both hands. Uniform pressure can be applied to the vertical cell by hand.

NOTE 6 -- In order to avoid parallax errors in obtaining readings, the operator's eyes should be at the same elevation as the level of the water in the capillary tube.

NOTE 7 -- If a significant sudden drop in the water column occurs just after starting the test and the water level continues to drop rather rapidly, this indicates water leakage, and the test should be repeated. If the sudden drop occurs in the first 20 to 30 seconds and then stops, this indicates that the porosity of the surface is high, and the test should be conducted in another location.

NOTE 8 -- When removing the cell from the surface, the operator should be careful not to spill the water on the surface if another test is scheduled in a nearby location.

8. Interpretation of Results

8.1 Use Table E-1 to interpret the results. These values were taken from concrete slabs prepared in the laboratory using the same concrete with different types of penetrating sealers applied to their surfaces and tested under different conditions.

Table E-1. Categories of sealer effectiveness.

Column Drop (cm) in 4 Minutes	Column Drop (cm) in 10 Minutes	Category
0 - 1	0 - 2	Good Sealer
1 - 2	2 - 4	Moderate Sealer
> 2	> 4	Poor Sealer

9. Report

9.1 The report shall consist of at least the following:

- 9.1.1 Location of test or identification of specimen,
- 9.1.2 Type of sealer and date of its application,
- 9.1.3 Specimen history or recent environmental field conditions,
- 9.1.4 Conditions at time of test,
- 9.1.5 Preconditioning of test area (if used),
- 9.1.6 Number of tests conducted for each case, and
- 9.1.7 Water drop readings and category.

10. Precision

10.1 *Precision* -- The single-operator within-lab standard deviation has been found to be 0.2 cm (1 s). Therefore, results of two properly conducted tests by the same operator on the same material should not differ by more than 0.6 cm (d2 s).

10.2 *Bias* -- The procedure in this test method for evaluation of penetrating sealers by water column drop has no bias because water column drop is defined only in terms of this test method.

11. Keywords

11.1 Bridges, concrete, in situ testing, nondestructive testing, penetrating sealers, water absorption.

Appendix F

Standard Test Method for Chloride Content in Concrete Using the Specific Ion Probe

Standard Test Method for

TOTAL CHLORIDE CONTENT IN CONCRETE USING THE SPECIFIC ION PROBE

1. Scope

1.1 This method covers the determination of the total chloride content in portland cement concrete structures using the prescribed sampling procedure in conjunction with a chloride specific ion probe.

1.2 The values stated in SI units are to be regarded as the standard.

1.3 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. Summary of Test Method

2.1 A 3.0-g concrete drilled powder sample is digested in 20 ml of digestion solution and stabilized by the addition of 80 ml of stabilizing solution. Millivolt readings taken for the final solution using a chloride specific ion probe are converted mathematically into

equivalent total percent chloride content or chloride content in pounds per cubic yard of concrete.

3. Significance and Use

3.1 Total chloride content in concrete structures is used to evaluate the degree of chloride contamination and indicates the probability of reinforcement corrosion.

4. Interferences

4.1 This test procedure is unsuitable for determining the chloride contents of concretes containing mineral aggregates having significant quantities of pyrite.

4.2 The operational response of the specific ion electrode is interfered with by the presence of OH^- , S^- , Br^- , I^- , and CN^- in the measured solution and is based on the electrode manufacturer's allowable chloride interference ratio.

5. Apparatus

5.1 *Impact Hammer* -- A heavy-duty rotary impact hammer is used. Figures F-1 through F-3 present the details of the drilling stop gage to be mounted on the impact hammer.

5.2 *Bit* -- A vacuum bit with a diameter equivalent to 1.5 times the maximum aggregate size in the concrete is used. A 28.6-mm (1 1/8-in.) bit size is recommended for a maximum coarse aggregate size of 19 mm (3/4 in.). The bit grinds the concrete to a fineness of 99% passing the #20 (850 μm) sieve. The finely ground powder is drawn into the collection unit through a coaxial hole in the bit.

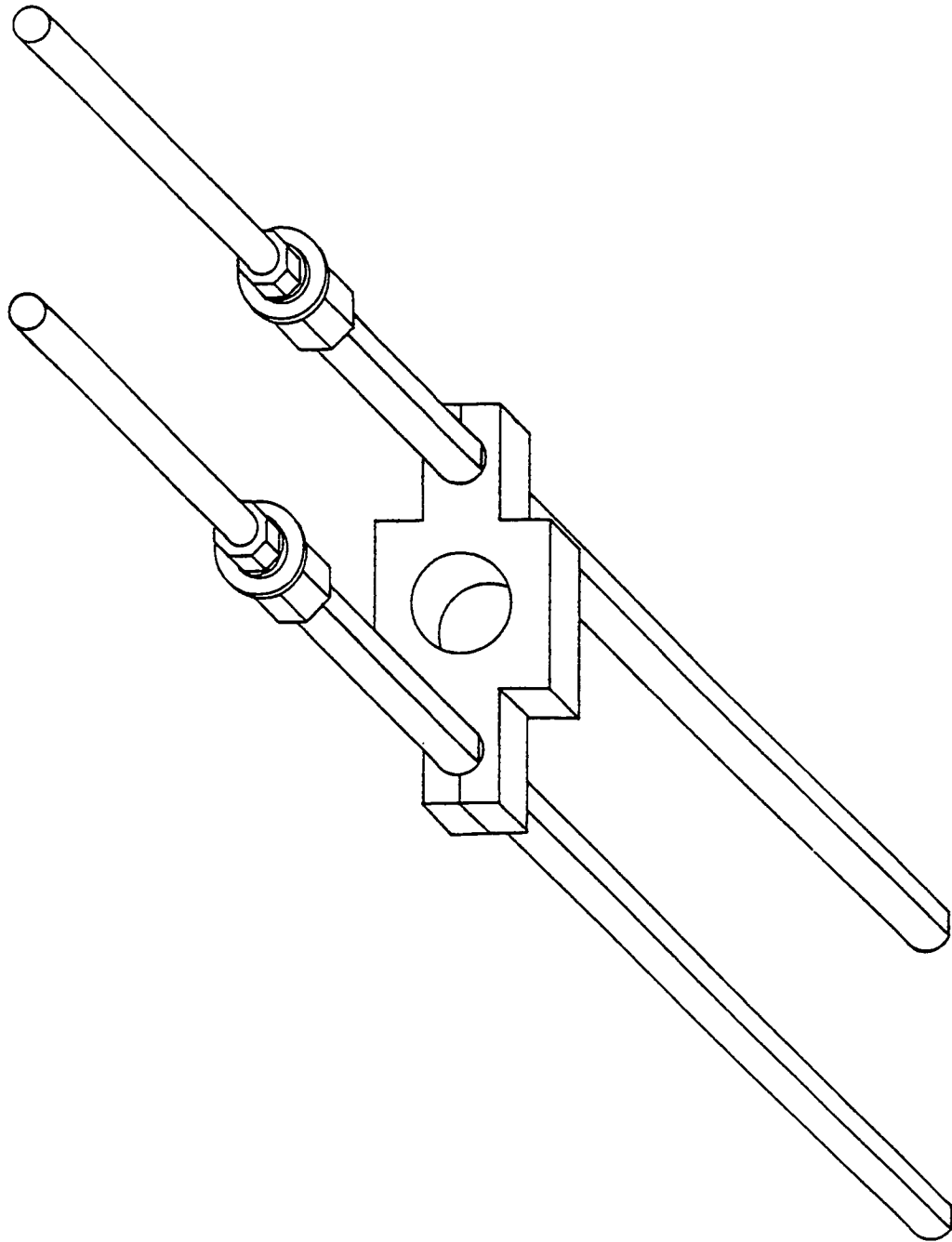
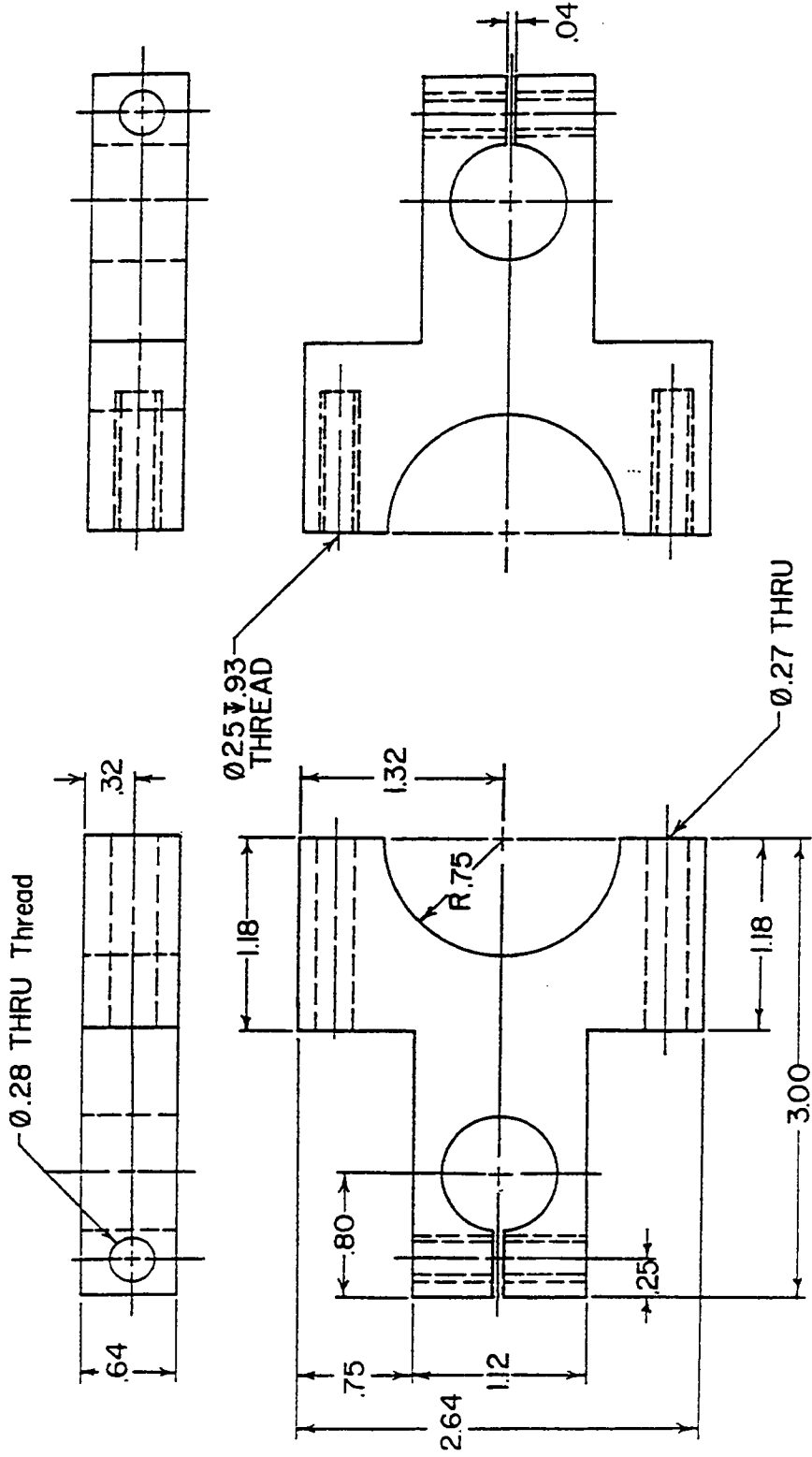
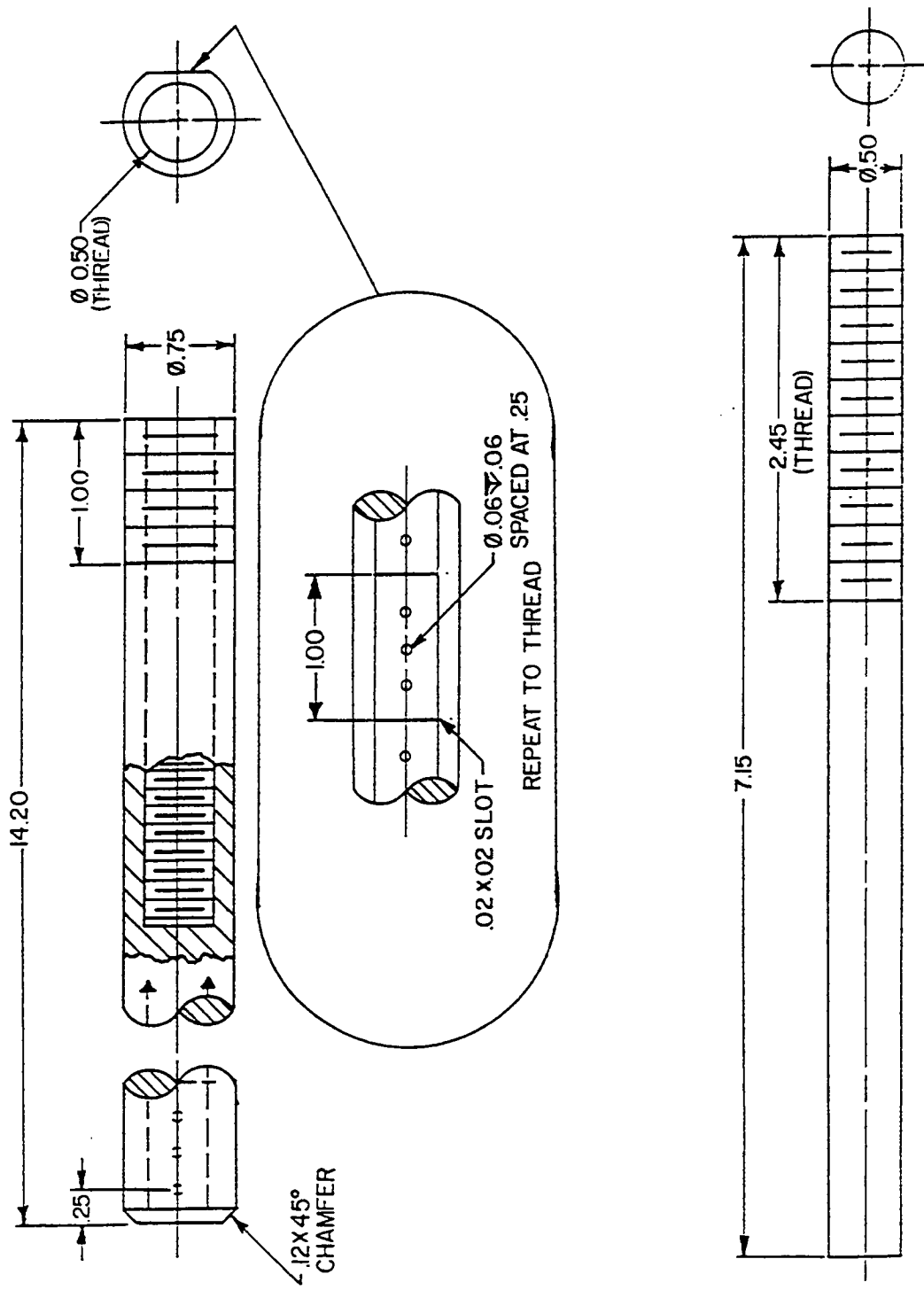


Figure F-1. Overall view of the drilling stop gage to be mounted on the impact hammer.



1 in. = 2.54 cm

Figure F-2. Details of the split ring collar for the drilling stop gage.



1 in. = 2.54 cm

Figure F-3. Details of the depth rods for the drilling stop gage.

5.3 *Sample Collection Unit* -- A cylindrical Plexiglas® vacuum sample collection chamber is connected to the carbide bit through a vacuum hose. Vacuum is provided by a 2.25-HP wet/dry shop vacuum unit. Figures F-4 through F-8 present the details of the sample collection unit that is to be attached to the wet/dry shop vacuum unit.

5.4 *Filter* -- An inexpensive coffee filter works best to collect the powdered concrete samples in the Plexiglas® chamber.

5.5 *Plastic Containers* -- 30-ml plastic containers are needed to store the concrete samples collected in the coffee filter.

NOTE 1 -- A parts list for the sampling unit is presented in Appendix 1.

5.6 *Weighing Papers* -- 76- x 76-mm (3- x 3-in.) pieces of treated paper are needed for weighing the powdered concrete samples.

5.7 *Balances* -- An electronic balance sensitive to 0.1 g is needed to weigh the specimens. An electronic balance sensitive to 0.0001 g is needed to weigh sodium chloride for the calibration and stabilizing solutions.

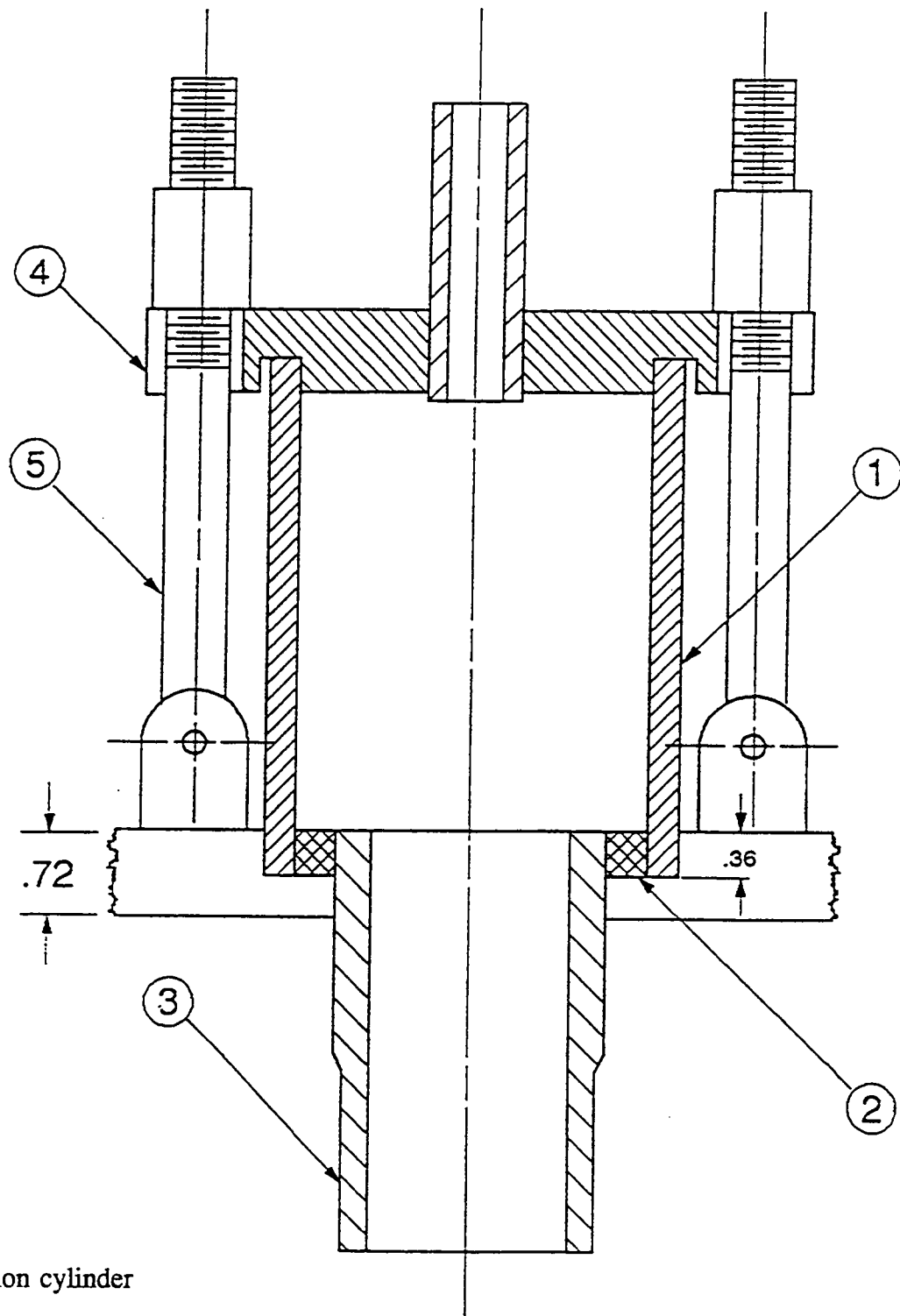
5.8 *Specific Chloride Ion Electrode* -- The specific ion electrode used to measure the concentration of chlorides present in solution must conform to the following specifications:

5.8.1 Self-contained unit.

5.8.2 Concentration measuring range 1 m to 5×10^{-5} m, or 35, 500 to 1.8 ppm.

5.8.3 The electrode resistance shall be less than 1.0 megohm.

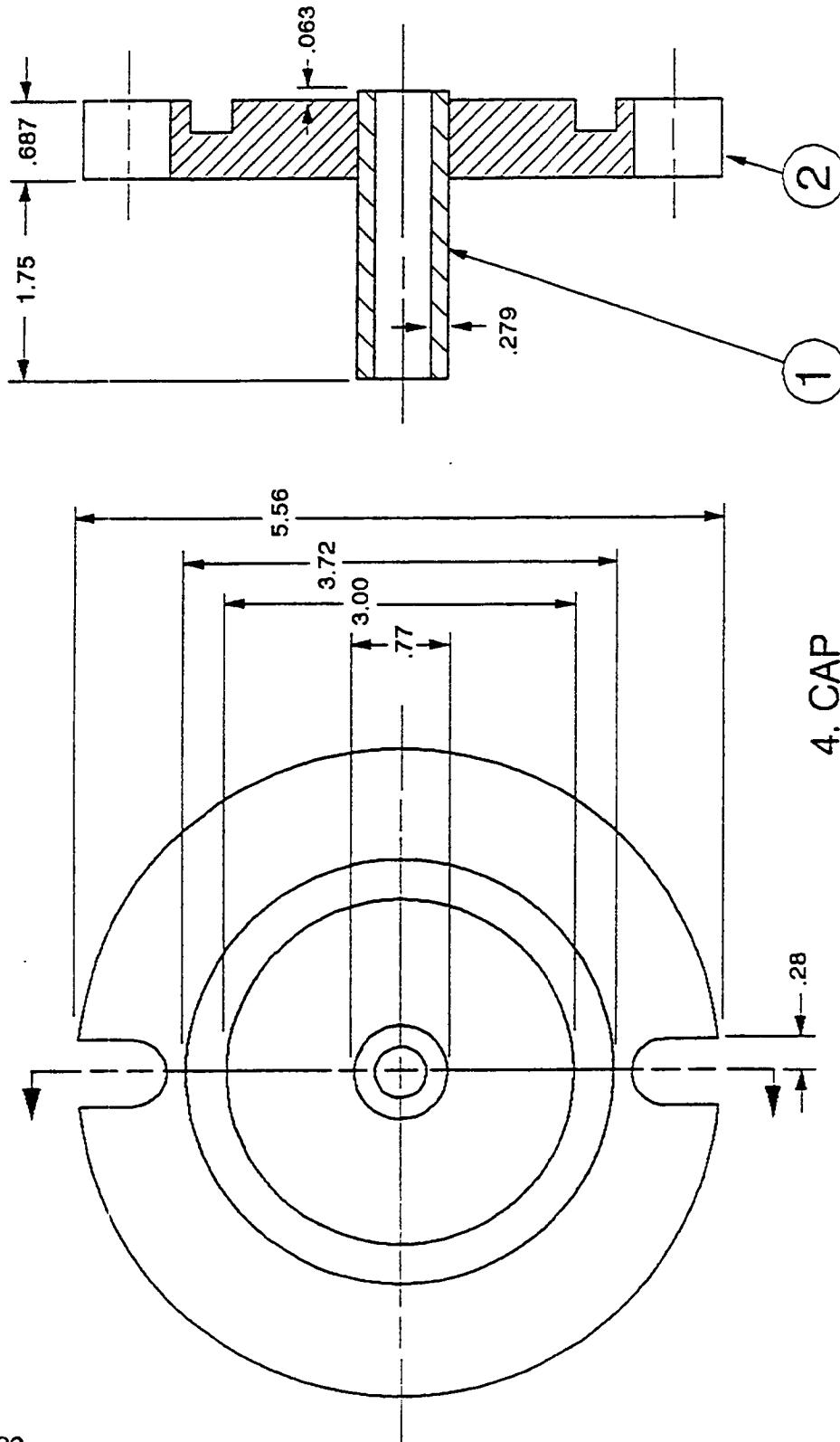
5.8.4 Reproducibility shall be $\pm 2\%$.



- 1. Collection cylinder
- 2. Ring
- 3. Vacuum cylinder
- 4. Cap
- 5. Clamp

1 in. = 2.54 cm

Figure F-4. Sample collection unit.

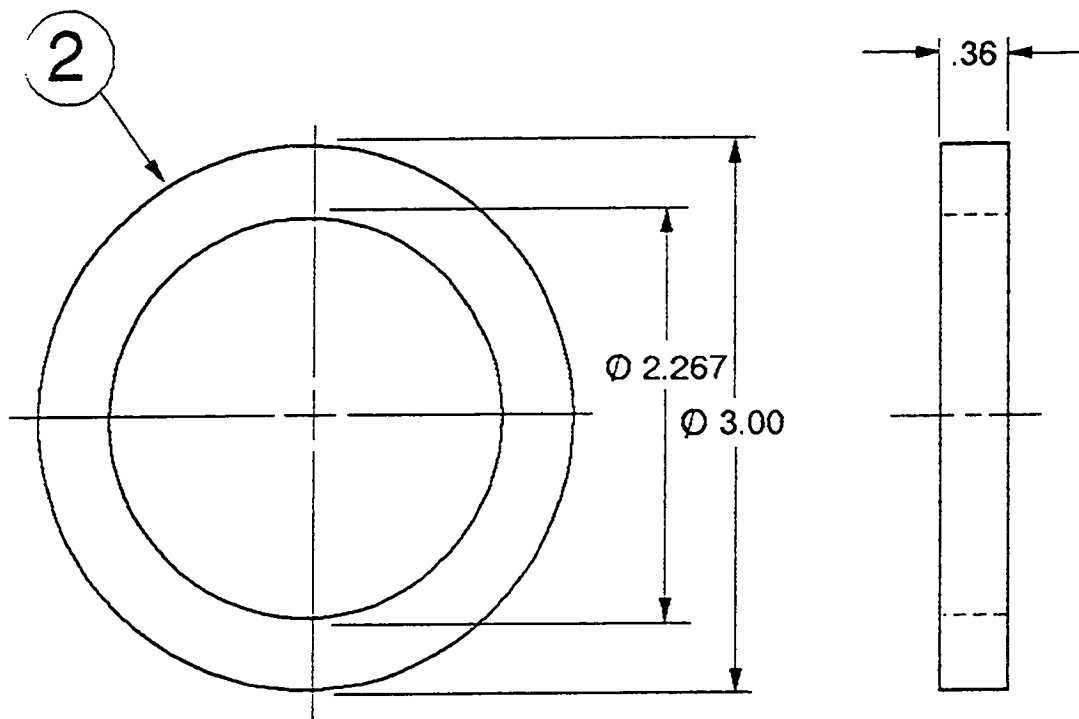


4. CAP

NAME	REQ'D	MATERIAL
1. ENTRY CYLINDER	1	PLEXIGLASS
2. CAP BASE	1	PLEXIGLASS

1 in. = 2.54 cm

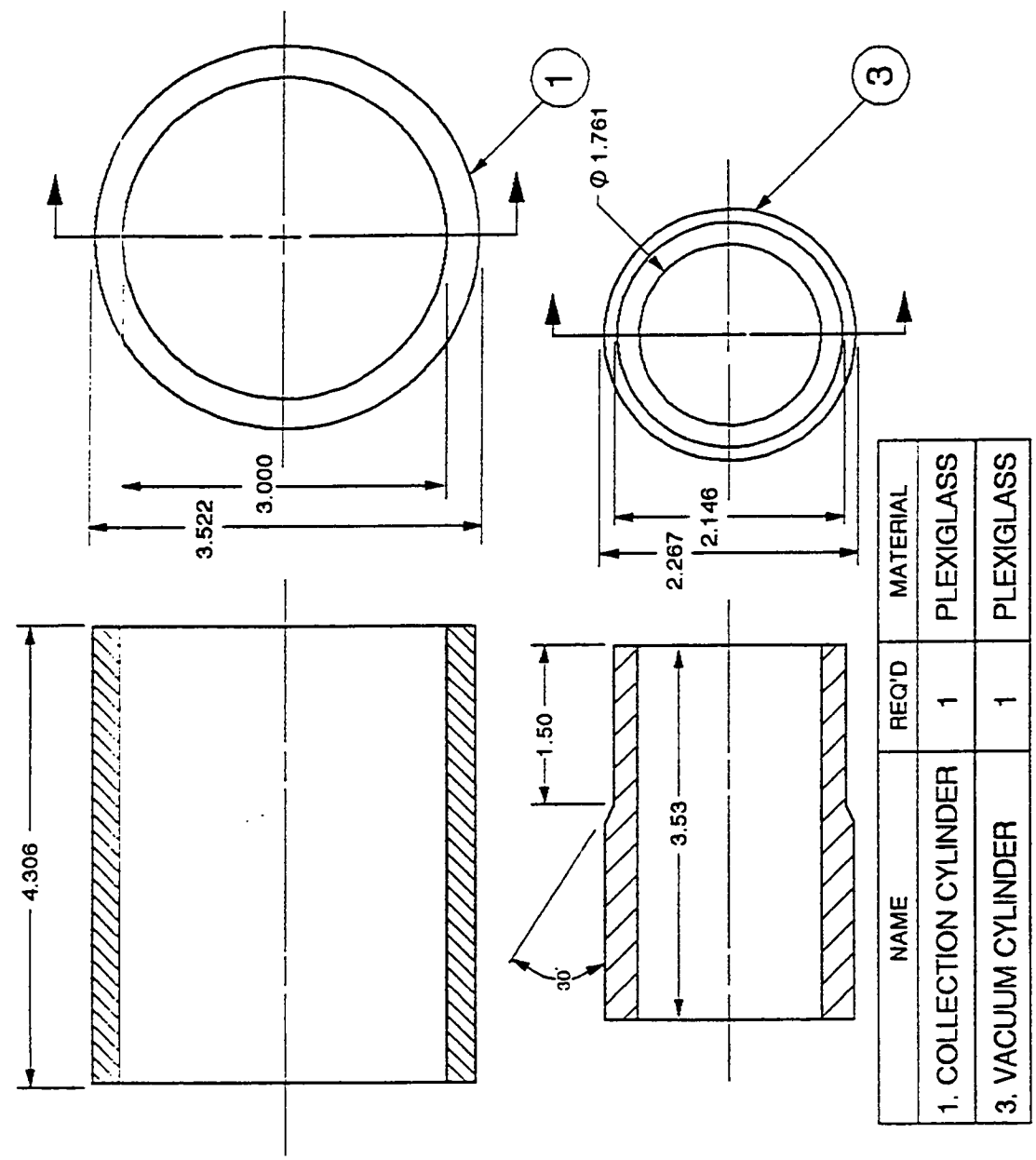
Figure F-5. Sample collection unit--details of cap.



NAME	REQ'D	MATERIAL
2. RING	1	PLEXIGLASS

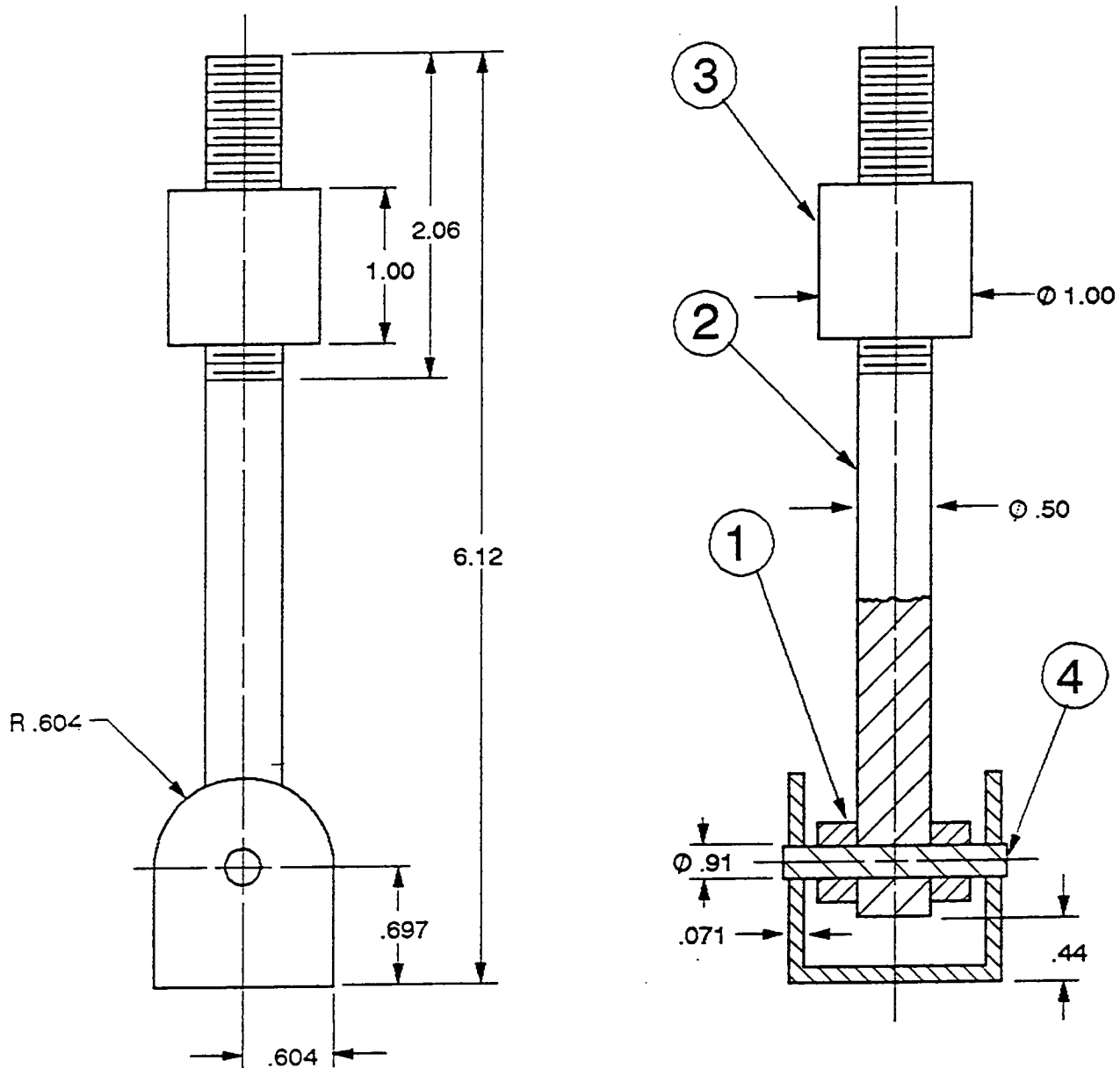
1 in. = 2.54 cm

Figure F-6. Sample collection unit--details of ring.



1 in. = 2.54 cm

Figure F-7. Sample collection unit--details of collection cylinder and vacuum cylinder.



5. CLAMP -- 2 REQ'D

NAME	REQ'D	MATERIAL
1. SPACER	2	RUBBER
2. SHAFT	1	STEEL
3. NUT	1	STEEL
4. SPRING PIN	1	STEEL

1 in. = 2.54 cm

Figure F-8. Sample collection unit--details of clamp.

5.8.5 Operating temperature range shall be 10 to 100°C.

NOTE 2 -- A suitable specific ion electrode has been found to be the Orion Combination Chloride Electrode Model 96-17B with chamber filling solution number 900017 (for use with chloride solution concentrations up to 355 ppm).

5.9 *Multimeter* -- The multimeter used in the procedure must conform to the following specifications:

5.9.1 Maximum common mode voltage shall be 1500 VDC or peak AC.

5.9.2 Resolution at minimal range shall be 100 μ V.

5.9.3 Accuracy shall be +(0.25% of the reading +1 digit).

5.9.4 Operating temperature range shall be 0 to 50°C (32 to 122°F).

5.9.5 Input impedance shall be greater than 2 M Ω .

NOTE 3--The Orion Model 250A is a suitable meter.

5.10 *Spatula* -- The spatula used to measure the powdered concrete samples shall be designed to extract material from the storage contained in small quantities.

5.11 *Digestion Solution* -- 20 ml is needed for each measurement.

5.12 *Stabilizing Solution* -- 80 ml of stabilizing solution is needed for each test.

5.13 *Calibration Solution* -- Five calibration solutions with 0.01-, 0.03-, 0.3-, 0.6-, and 1.25-percent chloride concentration based on a 3.0-g powdered concrete specimen are needed to calibrate the instrument.

6. Materials

6.1 *Sodium Chloride* -- Certified biological grade having an iodide concentration no greater than 0.0004%.

6.2 *Ethyl Alcohol* -- Standard grade.

6.3 *Acetic Acid* -- Glacial acetic acid having the following properties:

6.3.1 Specific gravity: 1.05.

6.3.2 Chloride content: ≤ 0.4 ppm.

6.3.3 Normality: 17.4.

6.4 *Isopropyl Alcohol* -- Standard grade having the following properties:

6.4.1 Density at 25°C: 0.781 to 0.783.

6.4.2 Boiling point: $82.3^{\circ} \pm 0.1^{\circ}\text{C}$.

6.5 *Distilled Water* -- Common distilled water produced in a laboratory still.

6.6 *Concrete Sample* -- The concrete sample to be analyzed shall be in powdered form, 99% passing the #20 (850 μm) sieve, as taken from the rotary impact hammer extraction procedure.

6.7 *Nalgene® Containers* -- Two sizes of wide-mouth Nalgene® containers are needed.

6.7.1 For calibration solutions: five, 40 ml.

6.7.2 For digestion solution: one, 125 ml.

7. Reagents

7.1 *Digestion Solution* -- The digestion solution is produced by combining acetic acid, isopropyl alcohol, and distilled water. Measure 940 g of distilled water in a 1-liter

container. Add 60 g of glacial acetic acid and 50 g of isopropyl alcohol to the distilled water. Thoroughly stir the solution. Dispense the solution into the 125-ml Nalgene® bottles in 20-ml volumes.

7.2 Calibration Solutions -- The calibration solutions are made by dissolving sodium chloride in distilled water. The concentration levels are 1.25, 0.6, 0.3, 0.03, and 0.01% by weight Cl⁻ based on a 3.0-g concrete sample. To achieve the respective concentration levels, mix the following quantities of sodium chloride with 1 liter of distilled water: 0.6169 g, 0.2961 g, 0.1481 g, 0.0148 g, and 0.0049 g. (This will result in solutions having chloride concentrations of 374, 180, 90, 9, and 3 ppm, respectively, which in turn correspond to the previously listed percentages by weight of concrete.) Dispense 20 ml into the 40-ml Nalgene® bottles and label each appropriately.

7.3 Stabilizing Solution -- The stabilizing solution is a dilution, standard chloride addition solution. It is prepared as follows: Place 0.1545 g of sodium chloride into 1 liter of distilled water. Add 40 ml of this solution to 960 ml of distilled water. This produces a 3.75 ppm chloride solution.

7.4 Temperature -- The temperature of the solution during testing significantly affects the determined chloride concentration. To minimize errors caused by temperature variations, ensure that the calibration, digestion, and stabilizing solutions are maintained at 72°F ± 2°F (22.2°C ± 1.1°C).

8. Sample Preparation

8.1 Obtain a sample from the concrete structure using the rotary impact hammer with depth gage and the carbide vacuum bit.

8.2 Using the sample collection unit, collect at least 10.0 g of powdered concrete in the filter. A 1 1/8-in. (2.86-cm)-diameter hole, 1/2-in. (1.27-cm) deep will yield approximately 25 g of powdered concrete sample.

8.3 Place the filter containing the sample in the 30-ml sample container.

9. Calibration of the Specific Ion Electrode

9.1 The measurement of the calibration solutions determines the response characteristics of the specific ion electrode in combination with the multimeter.

Mathematical regression of the responses produces a representative equation for the combination used to determine the chloride contents of powdered concrete samples. The minimum of five calibration solution concentrations is needed to adequately calibrate the equipment combination. Any alterations to equipment combinations shall require the formulation of a new calibration equation.

9.2 The calibration equation is determined as follows:

9.2.1 Calculate the \log_{10} of the ppm chloride concentrations of the calibration solutions, e.g., $\log_{10}(374 \text{ ppm Cl}) = 2.573$.

9.2.2 Perform a linear regression of the millivolt responses (X) versus the \log_{10} of the chloride concentrations of the respective calibration solutions (Y), producing an equation in the following format:

$$\log_{10} (\text{ppm Cl}^-) = \text{constant} + \text{coefficient} \times \text{mV} \quad (\text{F-1})$$

NOTE 4 – The constant and coefficient will vary with varying equipment combinations. However, the constant is numerically characterized by X.XXX and the coefficient is characterized by -0.0XXX, where X represents digits. The slope should be between 54 and 60 mV/decade at 25°C. If not, then consult the troubleshooting section of the manual for the electrode or the manufacturer.

9.2.3 The percent chloride is determined from the logarithmic inverse of the regression equation (equation F-1) minus the chloride added by the stabilizing solution, converted to percent chloride by weight of concrete, as follows:

$$\% \text{Cl}^- = [10^{(0.XXX + 0.0XXX \text{mV})} - 3 \text{ ppm}] (0.0333) \quad (\text{F-2})$$

9.3 Although unnecessary for calibration, daily measurements of the calibration solutions are recommended as a check on the operational integrity of the instrument combination. A deviation in the calibration solution millivolt response indicates a problem involving one or more of the following:

9.3.1 Contamination of calibration solutions.

9.3.2 Contamination of the internal filling solution of the specific ion electrode.

9.3.3 Defective specific ion electrode.

9.3.4 Defective multimeter.

9.4 The percent chlorides is based on a 3.0-g powdered concrete sample, and the ppm chloride is the resulting concentration of a 100-ml solution. The allowable deviations of the specific ion electrode are presented below:

Concentration (%Cl)	Allowable Deviation (mV)
1.25 (374 ppm)	1.5
0.60 (180 ppm)	1.5
0.30 (90 ppm)	1.5
0.03 (9 ppm)	1.5
0.01 (3 ppm)	4.5

9.5 The calibration equation shall be based on the data from ten calibration runs, each consisting of measurements of the five calibration solutions. The instrument shall be switched off between each of the ten runs.

10. Procedure

10.1 Place 20 ml of the digestion solution in a clean, dry 125-ml plastic bottle.

10.2 Remove the powdered concrete sample from the filter and return it to the 30-ml storage container. Shake and stir the powdered concrete sample to provide a uniform sample, and weigh 3.0 ± 0.1 g on a weighing paper.

10.3 Place the concrete sample in the bottle containing the 20-ml digestion solution, place the cap on the bottle, and shake vigorously to suspend the powder in the solution. Let

the powder digest for 3 minutes. The bottle may be opened after shaking to relieve pressure from effervescence due to the reaction, and then closed immediately.

10.4 Carefully remove the bottle cap and add 80 ml of the stabilizing solution (a 3.75-ppm chloride solution).

10.5 Replace the cap and shake vigorously for 1 minute.

10.6 Remove the bottle cap and place the electrode to a depth of mid-height in the extraction solution.

10.7 Wait 3 minutes and record the stable millivolt reading. The millivolt reading is stable when it oscillates by less than ± 0.5 mV.

10.8 After each millivolt measurement, the electrode shall be rinsed clean with distilled water.

11. Calculations -- Calculate the predicted percent chloride content in the sample as follows:

$$\text{Predicted \%Cl} = (A + B y) \quad (\text{F-3})$$

in which:

$$y = [10^{(e + D \text{ mV})} - 3] (0.0333) \quad (\text{F-4})$$

and

A = a unique value for a combination of a probe and a meter determined from the relationship between a standard titration method and this rapid method; a typical value is 0.8984

B = a unique value for a combination of a probe and a meter determined from the relationship between a standard titration method and this rapid method; a typical value is 1.165

C and D = The constants determined in the calibration step (see section 9); typical values are 2.855 and -0.01835, respectively.

mV = potentiometer reading, mV

12. Report -- Report the predicted percent chloride content to the third decimal (thousandth of a percent). (Or, in lb/yd³ to the first decimal (tenths of a lb/yd³) by multiplying by the conversion factor 39.15.)

13. Precision and Bias

13.1 *Precision* -- The single-operator within-laboratory standard deviation was found to be 1.91 mV (1 s). Therefore, the results of two properly conducted tests by the same operator on the same material should not differ by more than 5.40 mV (d2 s).

13.2 *Bias* -- The test method was found to have no bias in comparing experimental results with reference values determined in accordance with standard laboratory titration procedures (AASHTO T 260-84).

14. Significance of Results

This test method permits determination of the chloride contamination level of concrete structures as a function of depth. Using the values obtained, the chloride diffusion constant

can be calculated and the time to corrosion of embedded steel reinforcement can be estimated.

15. Keywords

12.1 Chloride content, chloride profile, concrete bridges, on-site testing, reinforcement corrosion, specific ion probe.

Appendix 1

Sampling Unit Parts List

1. 2.25 HP wet/dry shop vacuum
2. 3- to 4-ft Tygon tubing, clear 3/4-in. O.D., 1/2-in. I.D., 1/8-in. wall thickness.
3. Fabricated sample collection unit (see Figures F-5 through F-8).
4. 1 1/8-in. diameter Heller carbide vacuum draw drill bit, 13-in. long, Model 14825T.
5. Drill bit adapter, Heller, to match bit and rotary hammer.
6. Vacuum head adapter, 1 1/8 in., Heller Model 14869W.
7. Heavy-duty 1 1/2-in. rotary hammer with carrying case, Milwaukee, Model 5347.
8. Split ring collar (see Figure F-2).
9. Depth rods, two (see Figure F-3).

Appendix G

Standard Test Method for Indication of Relative Concrete Permeability by Surface Air Flow

Standard Test Method for

INDICATION OF RELATIVE CONCRETE PERMEABILITY BY SURFACE AIR FLOW

1. Scope

1.1 This test method covers the determination of the rate of flow of air out of the surface of concrete under an applied vacuum. The test method can be used as an indication of relative concrete permeability, but does not determine a rational permeability coefficient.

1.2 The surface air flow (SAF) measured by this test method is independent of the dimensions of the concrete, provided that at least 3 in. (75 mm) of clearance are provided to the nearest edge of the concrete under test.

1.3 The method is applicable to both laboratory specimens and field structures.

1.4 The values stated in inch-pound units (or cgs units) are to be regarded as the standard where inch-pound units (or cgs units) are given first, followed by SI units; where only SI units are given, or where SI units are given followed by inch-pound units (or cgs units), the SI units are to be regarded as standard.

1.5 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is

the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. Summary of Test Method

2.1 The method is based on measurement of the rate of air flow into a vacuum plate placed on the surface of concrete under a vacuum of approximately 125 mm of Hg (16.6 kPa) absolute pressure. A vacuum plate fitted with a soft rubber gasket is held in contact with the concrete surface, and a vacuum is applied. The rate of air flow is measured using a gas flowmeter. The rate of air flow (in ml/minute) is compared with typical values, and the relative air permeability is expressed in a qualitative manner as low, moderate, or high.

3. Significance and Use

3.1 This method may be used to indicate the relative permeability of the near-surface portion of a concrete specimen or member. It can be used to indicate the relative permeability of the concrete to ingress of chloride ions, water, or gases.

3.2 This method is applicable in both field and laboratory provided that the test surface is flat and that the center of the test area is no closer than approximately 3 in. (75 mm) from the edge of the specimen or member.

4. Interferences

4.1 Moisture content of the concrete can affect the test results. It may be necessary to pre-dry the test area prior to performing the test. This will normally be necessary only in

field applications where concrete has been exposed to rainwater for a significant period of time prior to performing the test. Drying can be accomplished by heating of the test area until a temperature between 250°F and 300°F (120°C and 150°C) is reached on the surface of the concrete. The concrete is then allowed to cool for 10 minutes prior to performing the test.

NOTE 1 -- If the concrete surface is only temporarily exposed to moisture (such as during a brief summer shower) and is allowed to dry under warm natural conditions for 2 hours or more, artificial drying may not be required.

4.2 Reinforcing steel may affect test results if cover is thinner than approximately 0.5 in. (12 mm). If such low cover is predominant over the area to be tested, reinforcing steel mats should be mapped out and readings obtained in locations not directly above reinforcing bars.

4.3 The test can be sensitive to surface roughness. Tests cannot be carried out on grooved or tyned (skid-textured) surfaces. Very rough or weathered surfaces may also pose problems. If the vacuum obtained during testing is 10 mm of Hg (1.3 kPa) less than the vacuum obtained against a metallic plate, then test results are questionable.

5. Apparatus

5.1 Testing apparatus for laboratory application of this method is shown schematically in Figure G-1. It consists of a gasketed vacuum plate, connector hose, flow meter, manometer, and vacuum pump. The configuration shown in Figure G-1 is not meant to

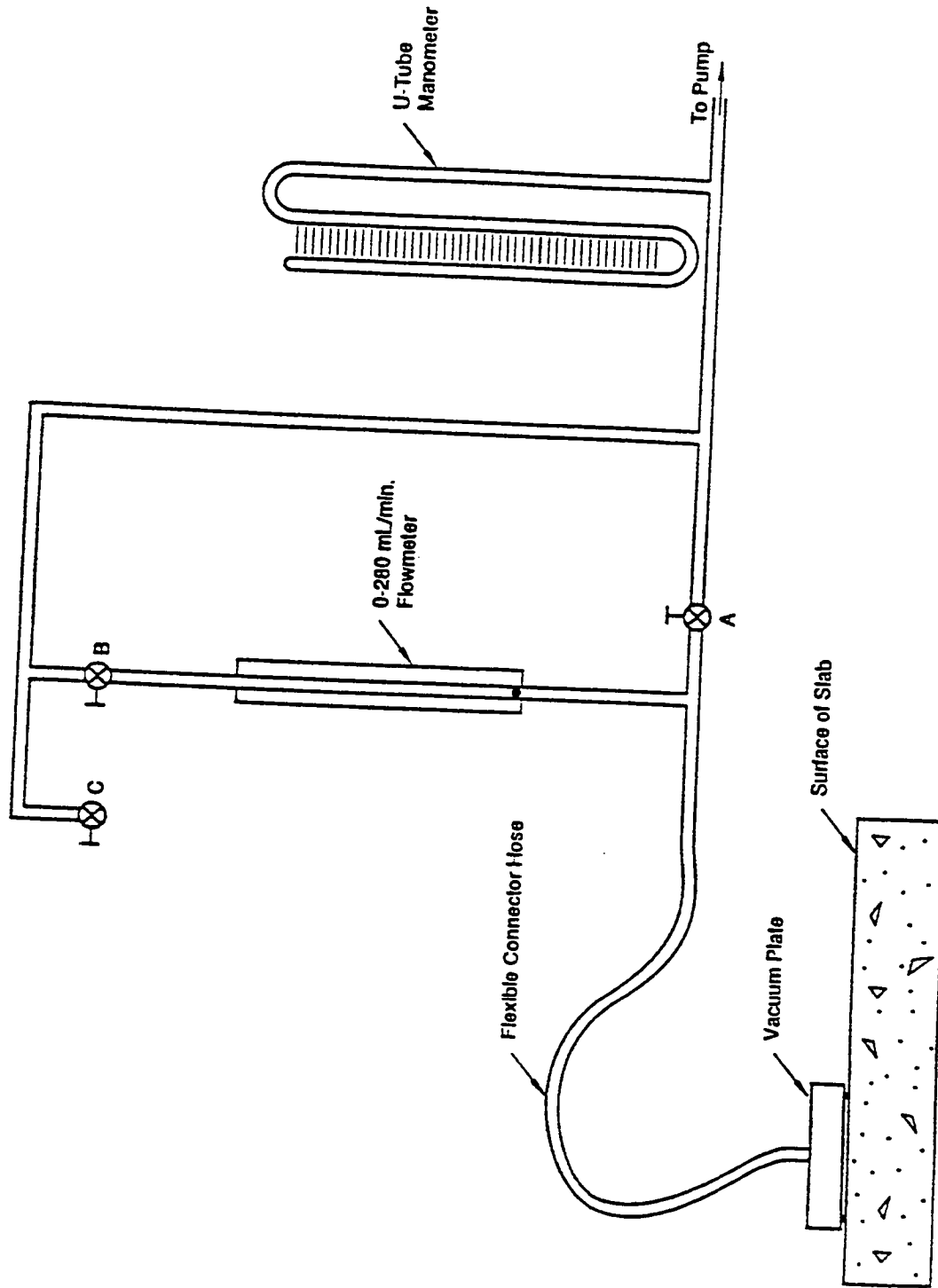


Figure G-1. Schematic drawing of the laboratory SAF device.

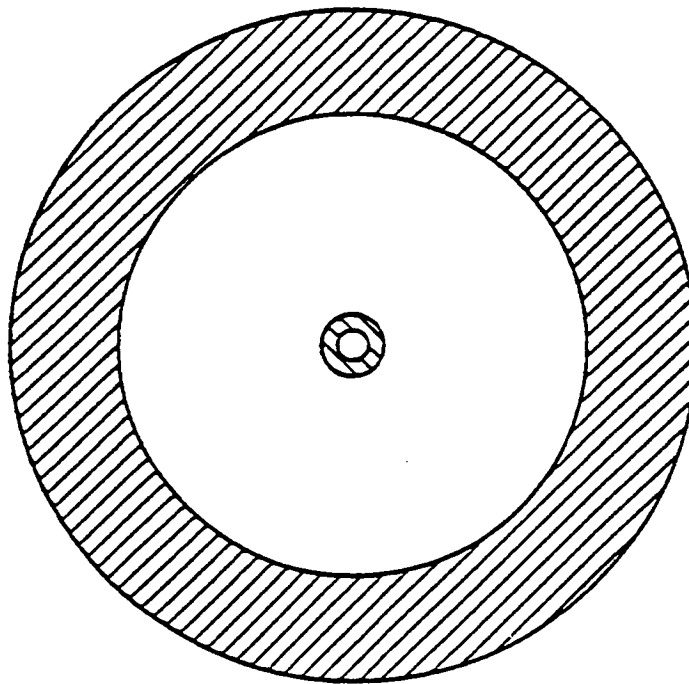
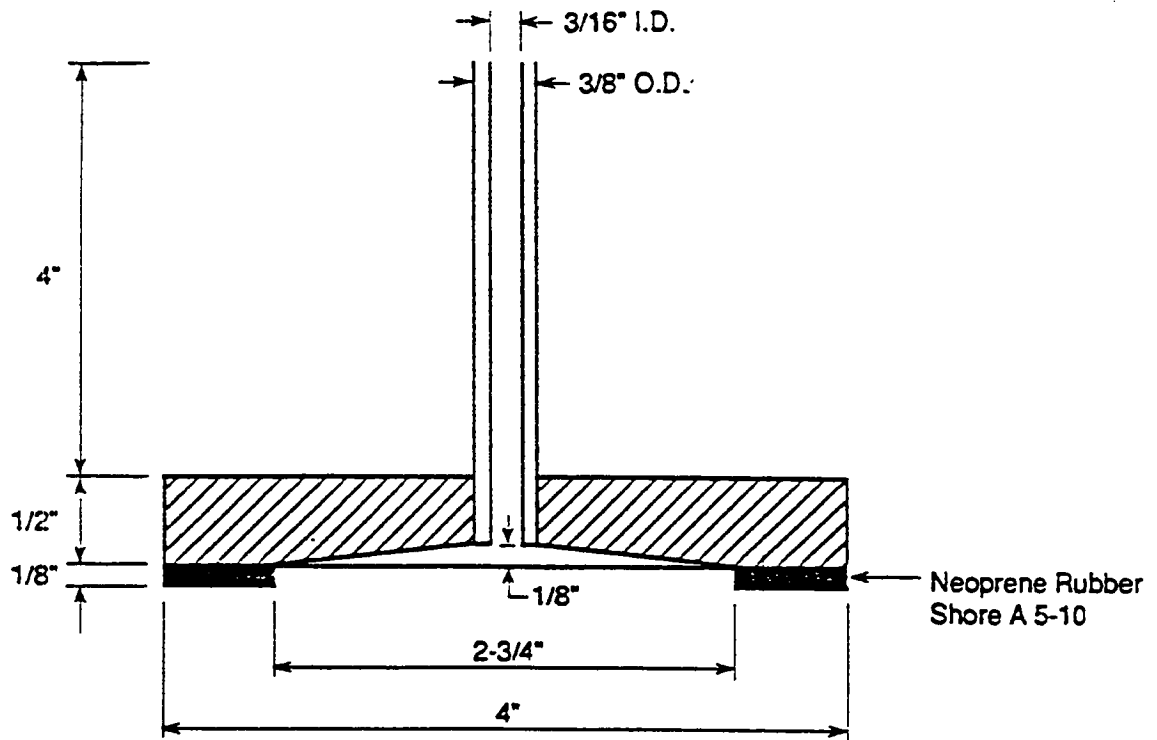
restrict the types or arrangements of components. For field work, portable, packaged systems may be constructed using electronic versions of the components shown in Figure G-1. However, for compatibility with this standard, the dimensions of the vacuum plate (Figure G-2) and the operating pressures must be as specified herein. A portable field device has been designed and tested¹ (see Figure G-3).

5.1.1 *Vacuum Plate* -- The vacuum plate shall be constructed of aluminum, steel, or other rigid impermeable material and have the dimensions shown in Figure G-2. A soft rubber gasket (shore hardness A 5-10) is affixed to the outer periphery of the plate with silicone adhesive.

5.1.2 *Flowmeter* -- The flowmeter shall be capable of operating within the range of approximately 0 to 280 ml/minute with a resolution of 5 ml/minute.

NOTE 2 -- Tapered rotameter-type flowmeters with spherical floats are typically supplied with a logarithmic scale. In this case, resolution will improve at the lower ends of the scale, and it may be readable to 2 ml/minute or less at flows less than about 40 ml/minute.

NOTE 3 -- Flowmeters operating on principles different from spherical float rotameters (such as electronic mass flowmeters) may yield values of flow different from those obtained with a rotameter in the configuration shown in Figure G-1. In these cases, a linear relationship between rotameter and electronic flowmeter values must be established (*I*).



Note: 1 in. = 2.54 cm

Figure G-2. Details of vacuum plate.



Figure G-3. Prototype SAF device in operation on a bridge deck.

5.1.3 *Manometer* -- The manometer shall be used to record the differential pressure created across the test specimen. If a U-tube manometer is used, then units are expressed in millimeters of mercury above absolute vacuum (i.e., 0 mm Hg [0 kPa] represents a perfect vacuum). Some electronic manometers may read in millimeters of Hg *below* atmospheric, such that -760 mm Hg (-101 kPa) represents a perfect vacuum.

5.1.4 *Vacuum Pump* -- The vacuum pump shall be capable of maintaining a vacuum level of approximately 125 ± 5 mm of Hg (16.6 ± 0.6 kPa) absolute pressure. A single-head diaphragm type vacuum pump with an evacuation capacity of approximately 9.2 l/minute has been found to be adequate for this purpose.

6. Sampling

6.1 *Spacing between Measurements* -- The spacing between measurements should be consistent with the size of the member being investigated and the intended use of the measurements.

NOTE 4 -- A spacing of 3 ft (0.9 m) has been found to be satisfactory for evaluation of bridge decks. Measurements may be taken in a grid or random pattern. On support or substructure elements, geometry of the element may preclude the use of the same spacing as on decks. In these cases smaller or larger spacings may be appropriate.

7. Procedure

7.1 *Surface Preparation* -- The surface must be dry, clean, and free from grooves, cracks and irregularities, which could prevent obtaining a good vacuum seal. If significant

rainfall has occurred during the past 24 hours and the surface is now in a surface-dry condition, it may be necessary to spot-dry each test location selected. This can be accomplished by heating the surface to a temperature of between 250°F and 300°F (120°C and 150°C) by holding a heat source approximately 4 in. (100 mm) from the test surface until such time that the specified surface temperature has been reached. The surface should then be allowed to cool for a period of 10 minutes. The surface should then be brushed free of dust, dirt, or debris prior to testing.

NOTE 5 -- To determine whether it is indeed necessary to pre-dry the test area, tests should be carried out on one or two selected locations before and after drying. If results are essentially the same (within \pm 5 ml/minute) then pre-drying is not necessary on the remaining test locations.

7.2 Leak Check Test -- The vacuum plate should be placed against an impermeable surface, such as a rigid metallic or polymeric sheet. The vacuum pump is then activated and the vacuum level is monitored. After a period of 30 seconds the vacuum should be within the range of 125 ± 5 mm of Hg (16.6 ± 0.6 kPa) absolute pressure. Flow should then be directed through the flowmeter, which should read less than 5 ml/minute. If these conditions are not met, a leak in the vacuum system is indicated, and the problem should be corrected before proceeding with the testing.

7.3 Determination of Air Flow Rate -- Place the vacuum plate on the surface to be tested. Ensure that Valves A, B, and C are in the closed position. Activate the vacuum pump and observe the vacuum level on the manometer. The vacuum should be stabilized at

125 ± 5 mm Hg (16.6 ± 0.6 kPa) absolute pressure within 15 seconds. Open Valve A and again observe the vacuum level over a period of 15 seconds. The vacuum should stabilize within the range of 125 ± 10 mm Hg (16.6 ± 1.2 kPa) absolute pressure (Note 6). Open Valve B and close valve A so that all flow is directed through the flowmeter. After a period of 15 seconds, record the reading obtained on the flowmeter (Note 7). If the reading is off scale, this should be noted and taken as a valid reading if the specified vacuum levels have been achieved and the leak test has indicated the system to be functional.

NOTE 6 – Vacuum levels less than 115 mm Hg (15.3 kPa) absolute pressure are generally indicative of cracks, crazing, or other irregularities in the test surface, which prevent obtaining the specified vacuum. They may also be indicative of extremely permeable concrete. However, this condition cannot generally be distinguished from the effects of cracks, etc.

NOTE 7 – There may be some amount of pulsing in the flowmeter float depending on characteristics of the vacuum pump used. The operator should note the upper and lower limits of the pulses and record the midpoint of the range. In most instances the range is not more than about 5 ml/minute.

8. Interpretation of Results

8.1 Use Table G-1 to interpret the results. These values were taken from slabs prepared in the laboratory from concretes of various compositions.

9. Report

9.1 The report shall consist of at least the following:

9.1.1 Location of test or identification of specimen,

9.1.2 Specimen history or recent environmental field conditions,

- 9.1.3 Conditions at time of test,
- 9.1.4 Preconditioning of test area (if used),
- 9.1.5 Check test vacuum and flow rate, and
- 9.1.6 Test flow rate and category.

Table G-1. Categories of relative permeability.

Air Flow Rate (ml/minute)	Relative Permeability Indicated (Category)
0 to 30	Low
30 to 80	Moderate
over 80	High

10. Precision and Bias

10.1 *Precision* -- The single-operator, within-lab standard deviation has been found to be 3.6 ml/minute (1 s). Therefore, results of two properly conducted tests by the same operator on the same material should not differ by more than 10.2 ml/minute (d2 s).

10.2 *Bias* -- The procedure in this test method for measuring relative concrete permeability by SAF has no bias because the value of SAF is defined only in terms of this test method.

11. Keywords

11.1 Air flow, bridges, bridge decks, chlorides, concrete, in situ testing, nondestructive testing, permeability, vacuum.

Endnote

¹For details see: *Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion Volume 7: Method for Field Measurement of Concrete Permeability*. Strategic Highway Research Program, National Academy of Sciences.

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