SHRP-C-364

Mechanical Behavior of High Performance Concretes, Volume 4

High Early Strength Concrete

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Strategic Highway Research Program
National Research Council
Washington, DC 1993

SHRP-C-364
ISBN 0-309-05756-6
Contract C-205
Product No. 2014, 2023, 2024

Program Manager: *Don M. Harriott*
Project Manager: *Inam Jawed*
Production Editor: *Cara J. Tate*
Program Area Secretary: *Carina S. Hreib*

December 1993

key words:
ac impedance
admixture
aggregate
compressive strength
concrete mixture
creep
durability
elastic modulus
high performance concrete
permeability
portland cement concrete
  early strength
  high strength
shrinkage
tensile strength

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1.5M/NAP/1293

Acknowledgments

The research described herein was supported by the Strategic Highway Research Program (SHRP). SHRP is a unit of the National Research Council authorized by section 128 of the Surface Transportation and Uniform Relocation Assistance Act of 1987.

The research was conducted by a consortium among researchers at North Carolina State University (prime contractor), the University of Arkansas, and the University of Michigan. The team included Paul Zia, Shuaib H. Ahmad, and Michael L. Leming at North Carolina State University; John J. Schemmel and Robert P. Elliott at the University of Arkansas; and A. E. Naaman at the University of Michigan.

The late Robert E. Philleo served as a project consultant during the initial stage of the project. He helped in defining the criteria for high performance concrete and in setting the direction of the research. The research team benefited greatly from his many stimulating discussions and guidance.

The research team also received valuable support, counsel, and guidance from the Expert Task Group. The support and encouragement provided by Inam Jawed, SHRP program manager, are deeply appreciated.

Special recognition is given to New York Department of Transportation, North Carolina Department of Transportation, Illinois Department of Transportation, Arkansas State Highway and Transportation Department, and Nebraska Department of Roads. The cooperation and assistance given by the staffs of these five state transportation agencies were critical to the field installations of pavements to demonstrate the use of high performance concrete.

A group of materials producers and suppliers donated the materials used for the laboratory studies of high performance concrete. These industrial partners included Blue Circle Cement, Inc.; Pyrament/Lone Star Industries, Inc.; Martin Marietta Company; Memphis Stone and Gravel Co.; McClinton-Anchor; Arkhola Sand and Gravel; Monex Resources, Inc.; Fly Ash Products; Elkem Chemicals; Cormix; Dow Chemical; and W. R. Grace. Their generous support of this research program is deeply appreciated.

The authors of this report would also like to acknowledge the contributions of many collaborators who participated in the various phases of this research program. They include former and present graduate research assistants William K. Chi, Wayne L. Ellenberger, M. R.

Hansen, Aykut Cetin, Kristina M. Hanes, Johnny D. Waycaster, Randy Maccaferri, and Dena Firebaugh and former undergraduate laboratory assistants Randall A. Boyd, Joseph B. Taylor, Wesley N. Denton, and Harold D. Ingram at North Carolina State University. The collaborators at the University of Arkansas include former and present graduate research assistants James R. Powell, Vikas Arora, and Mike Callahan and former undergraduate laboratory assistant Memory Rodgers.

Opinions expressed in this report reflect the views of the authors, who should be held accountable for any errors and omissions.

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Preface

The Strategic Highway Research Program (SHRP) is a 5-year, nationally coordinated research effort initiated in 1987 at a cost of $150 million. This highly focused and mission oriented program originated from a thorough and probing study* to address the serious problems of deterioration of the nation's highway and bridge infrastructure. The study documented the need for a concerted research effort to produce major innovations for increasing the productivity and safety of the nation's highway system. Further, it recommended that the research effort be focused on six critical areas in which the nation spends most of the $50 billion used for roads annually and thus technical innovations could lead to substantial payoffs. The six critical research areas were as follows:

- Asphalt Characteristics
- Long-Term Pavement Performance
- Maintenance Cost-Effectiveness
- Concrete Bridge Component Protection
- Cement and Concrete
- Snow and Ice Control

When SHRP was initiated, the two research areas of Concrete Bridge Component Protection and Cement and Concrete were combined under a single program directorate of Concrete and Structures. Likewise, the two research areas of Maintenance Cost-Effectiveness and Snow and Ice Control were also combined under another program directorate of Highway Operations.

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* *America's Highways: Accelerating the Search for Innovation.* 1984. Special Report 202, Transportation Research Board, National Research Council, Washington, D. C.

Abstract

This report documents the laboratory investigations of the mechanical behavior and field trials of high performance concrete for highway applications. High performance concrete is defined as concrete with much higher early strength and greatly enhanced durability against freezing and thawing compared with conventional concrete. High early strength (HES) concrete is one of the three categories of high performance concrete investigated in this program. The objective is to obtain information on the mechanical behavior of HES concrete and to demonstrate its use under field conditions.

The laboratory investigation consisted of tests for both fresh or plastic concrete and hardened concrete. The plastic concrete tests included slump, air content, etc.; the results of these tests are presented in volume 2 of this report series, *Production of High Performance Concrete*. The hardened concrete tests included compression tests for strength amd modulus of elasticity, tension tests for tensile strength and flexural modulus, freezing-thawing tests for durability factor, shrinkage tests, rapid chloride permeability tests, tests for AC impedance, tests for concrete-to-concrete bond, and tests for bond between concrete and steel reinforcement.

The field experiments were conducted in five states that represented a wide variety of environmental and exposure conditions: New York, North Carolina, Arkansas, Illinois, and Nebraska.

The results of the laboratory work and field experiments indicated that HES concretes with enhanced frost resistance can be successfully produced in the laboratory and used in the field for highway pavements.

Executive Summary

This report documents laboratory investigations of mechanical behavior and field trials of high performance concrete for highway applications. High performance concrete (HPC) is defined as concrete with much higher early strength and greatly enhanced durability against freezing and thawing compared with conventional concrete. High early strength (HES) concrete is one of the three categories of high performance concrete investigated in this program. The objective is to obtain information on the mechanical behavior of HES concrete and to demonstrate its use under field conditions.

For the purpose of this program, HPC is defined in terms of certain *target* strength and durability requirements as shown below:

Category of High Performance Concrete	Minimum Compressive Strength	Maximum Water/ Cement Ratio	Minimum Frost Durability Factor
Very early strength (VES)			
Option A (with Type III cement)	2,000 psi (14 MPa) in 6 hours	0.40	80%
Option B (with Pyrament PBC-XT cement)	2,500 psi (17.5 MPa) in 4 hours	0.29	80%
High early strength (HES) (with Type III cement)	5,000 psi (35 MPa) in 24 hours	0.35	80%
Very high strength (VHS) (with Type I cement)	10,000 psi (70 MPa) in 28 days	0.35	80%

In the above definition, the target minimum strength should be achieved in the specified time after water is added to the concrete mixture. The compressive strength is determined from 4 x 8-in. (100 x 200 mm) cylinders tested with neoprene caps. The water/cement ratio (W/C) is based on all cementitious materials. The minimum durability factor of 80% should be achieved after

300 cycles of freezing and thawing according to ASTM C 666, procedure A. This is in contrast to a durability factor of 60% for conventional concrete.

So that the research results would be applicable to different geographical regions, four different types of coarse aggregate were selected for producing HES Concrete. They included crushed granite and marine marl from North Carolina, dense crushed limestone from Arkansas, and washed rounded gravel from Tennessee. These aggregates were used with local sand from the three states. The characteristics of all constituent materials used for producing HES concrete are described in detail in terms of their physical, chemical, and mineral properties. The normal laboratory mixing and batching procedures (ASTM C 192) were modified slightly to represent typical concrete dry-batch plant operationsmore closely.

The laboratory investigation consisted of tests for both fresh or plastic concrete and hardened concrete. The plastic concrete tests included slump, air content, etc.; and the results of these tests are presented in volume 2 of this report series, *Production of High Performance Concrete*. The hardened concrete tests included compression tests for strength and modulus of elasticity, tension tests for tensile strength and flexural modulus, freezing-thawing tests for durability factor, shrinkage tests, rapid chloride permeability tests, AC impedance tests, concrete-to-concrete bond tests, and tests for bond between concrete and steel reinforcement.

The field experiments were conducted in five states that represented a wide variety of environmental and exposure conditions: New York, North Carolina, Arkansas, Illinois, and Nebraska.

Based on the experience of the laboratory investigations and the field trials, the following conclusions were drawn:

1. Using conventional materials and equipment, but with more care than needed for conventional concrete, it is possible to produce in the laboratory, as well as in the field, HES concrete that will achieve a minimum compressive strength of 5,000 psi (35 MPa) in 24 hours. Such concrete was produced with a variety of aggregates including crushed granite, marine marl, dense crushed limestone, and washed rounded gravel.

2. By using insulation to trap the heat of hydration, the strength development of HES concrete can be accelerated to achieve a very early strength (VES) concrete with a strength of 2,000 psi (14 MPa) or more in 6 hours.

3. Because of a larger amount of Type III cement used in the HES concrete mixture along with a fast-acting accelerator and a relatively low W/C, the strength development of the concrete is much more rapid in the first 15 days than predicted by the current ACI Committee 318 recommendation (1993b) based on the conventional concrete.

4. Because the HES concrete in this study was kept moist only for the first 24 hours followed by air curing in the laboratory, the strength development of the small laboratory samples is very rapid during the first day and the subsequent rate of strength growth is greatly reduced. The same is true for the modulus of elasticity.

5. Since the design strength (i.e., 5,000 psi or 35 MPa) of HES concrete is within the range of conventional concrete, the mechanical behavior of HES concrete, such as the modulus of elasticity and the compressive and tensile strain capacities, is similar to that of conventional concrete. The modulus of elasticity, the flexural modulus, and the splitting tensile strength can all be predicted quite well by the ACI Code equations (1993b). The compressive strain capacity ranges from 1,500 to 2,000 microstrains, and the tensile strain capacity varies from 150 to 250 microstrains.

6. The stress-strain relationship of HES concrete is more nonlinear at 1 day than at later ages, and the modulus of elasticity is lower for concrete with softer aggregate (e.g., marine marl) or with aggregate that contains more moisture (e.g., washed rounded gravel).

7. At the design age of 1 day, both the strength and the elastic modulus of the HES concrete with latex are somewhat lower than those of the comparable concrete without latex due to a short curing time. However, the concrete with latex, which produced a compressive strain capacity of nearly 3,000 microstrains at maximum strength, is far more ductile than the concrete without latex, which showed a compressive strain capacity of 1,500 microstrains at maximum strength.

8. Even though its W/C is low, HES concrete should have an adequate amount of air entrainment to enhance its freeze-thaw resistance. The results of this investigation indicate that HES concrete will meet the stringent requirement of a durability factor of 80% (as compared with 60% for conventional concrete) after 300 cycles of freezing and thawing according to ASTM C 666, procedure A, if the concrete contains at least 5% entrained air.

9. Shrinkage of HES concrete follows the general trend of conventional concrete. The average shrinkage strain of HES concrete at 90 days ranges from 210 to 481 microstrains, depending on the type of coarse aggregate used. These values represent 30% to 70% of the ultimate shrinkage strain recommended by the ACI Committee 209 (1993a) for conventional concrete.

10. The normal procedure of the rapid chloride permeability test (RCPT) is to measure the total electrical charge (in coulombs) flowing through a vacuum-saturated concrete specimen in 6 hours. This measurement is regarded as an indication of chloride ion permeability of the concrete. HES concrete may exhibit high chloride permeability according to the RCPT since many additional ions introduced into the concrete by the various admixtures will cause the concrete to be more electrically conductive and make it appear to be more permeable than it really is.

11. The initial current (in amperes) flowing through the concrete specimen in the RCPT correlates consistently with the total charge measured in 6 hours. Therefore the initial current, which is an indirect measure of concrete conductance, can be used as an alternate measurement for the RCPT. The total testing time can thus be shortened by 6 hours.

12. The AC impedance test measures the total resistance (in ohms) of a concrete specimen. This test method is simpler and faster than the RCPT and has the potential to be used as a substitute for the RCPT. The best correlation between the two test methods is to express the inverse impedance (reciprocal of impedance) in terms of the initial current measured in the RCPT.

13. Concrete-to-concrete bond strength can be determined by a direct shear test. The HES concrete with crushed granite developed a nominal bond strength of 275 psi (1.93 MPa) with the normal North Carolina Department of Transportation (NCDO) pavement concrete. The HES concrete with marine marl developed a nominal bond strength of 350 psi (2.45 MPa) with the same NCDOT pavement concrete. These values are comparable to the corresponding value of 330 psi (2.31 MPa) obtained from the control test using the NCDOT concrete.

14. The concrete-to-steel bond tests indicated that the HES concrete with crushed granite as aggregate developed sufficient bond strength with steel to satisfy the ACI 318 requirement on development length.

15. The experience gained from the five field experiments indicates that the slump and air content of HES concrete are much more difficult to control in the field than in the laboratory. This is not unlike the case with conventional concrete.

16. At the various ages of the concrete, the strengths of the cores taken from the experimental pavements generally correlate well with the strengths of the control cylinders.

17. The results of the RCPT performed on the cores taken from the experimental pavement in North Carolina confirm the results obtained from the specimens prepared in the laboratory.

18. The temperature history of the control cylinders cured in an insulated curing box in the field generally corresponds very well with the temperature history of the pavement.

19. To produce the HES concrete in the field, thorough mixing of the concrete is critical. Batch size should be limited to no more than one-half to two-thirds of the rated capacity of the ready-mix concrete truck.

20. To optimize the performance of any field installation, preconstruction meetings should be held with contractors, concrete suppliers (including batch plant operators), and appropriate personnel of highway agencies.

21. Laboratory and field trial batches should be produced in sufficient numbers to confirm the mixture proportions, batching sequence, and workability of the concrete.

1

Introduction

SHRP's research on mechanical behavior of high performance concretes had three general objectives:

1. To obtain needed information to fill gaps in the present knowledge;
2. To develop new, significantly improved engineering criteria for the mechanical properties and behavior of high performance concretes; and
3. To provide recommendations and guidelines for using these concretes in highway applications according to the intended use, required properties, environment, and service.

Both plain and fiber-reinforced concretes were included in the study. The research findings are presented in a series of six project reports:

Volume 1 *Summary Report*

Volume 2 *Production of High Performance Concrete*

Volume 3 *Very Early Strength (VES) Concrete*

Volume 4 *High Early Strength (HES) Concrete*

Volume 5 *Very High Strength (VHS) Concrete*

Volume 6 *High Early Strength Fiber-Reinforced Concrete (HESFRC)*

This volume is the fourth of these reports. The readers will notice a certain uniformity in format and similarity in many general statements in these reports. This feature is adopted intentionally so that each volume of the reports can be read independently without the need to cross reference to other reports in the series.

1.1 Definition of High Early Strength Concrete

High early strength (HES) concrete is one of the three categories of high performance concrete investigated in this research program. In volume 2 of this report series, the strength and durability criteria were defined for each of the three categories of high performance concrete.

For HES concrete, a minimum compressive strength of 5,000 psi (35 MPa) is required in 24 hours with a maximum water/cement ratio (W/C) of 0.35. It must also achieve a minimum durability factor of 80% at 300 cycles of freezing and thawing according to ASTM C 666, procedure A. These criteria were adopted after several important factors were considered with respect to the construction and design of highway pavements and structures. The rationale for the selection of the various limits is as follows:

The use of a time constraint of 24 hours for HES concrete is intended for projects with accelerated construction schedules but without critical conditions, such as congested traffic in urban areas. The strength requirement of 5,000 psi (35 MPa) is selected to provide a class of concrete that would meet the need for accelerated construction of pavements and bridges. Since HES concrete is intended for highway applications where exposure to frost must be expected, it is essential that the concrete be frost-resistant. Thus it is appropriate to select a maximum W/C of 0.35, which is relatively low in comparison with that of conventional concrete. With a low W/C, concrete durability would be improved in all exposure conditions. Since HES concrete is expected to be in service at 1 day or less, the W/C selected might provide a discontinuous capillary pore system at about that age, as suggested by Powers' work (1959).

The selection of an appropriate measure for frost durability is debatable and subjective. It is recognized that ASTM C 666, procedure A, which involves freezing and thawing in water, is already a severe test. Therefore a durability criterion need not be unduly conservative. On the other hand, if high performance concrete is to provide enhanced durability, it can be argued that higher standards are required. Since frost durability of concrete as measured by ASTM C 666 (Procedure A) is highly dependent on the air void system, and since freezing low-permeability concrete at the very high rate required in the test procedure would tend to discriminate against concrete with low W/C, the selected durability factor of 80% at 300 cycles of freezing and thawing is considered appropriate. This is in contrast to a durability factor of 60% commonly expected of quality conventional concrete according to ASTM C 666.

1.2 Potential Applications of HES Concrete

HES concrete is probably the most versatile construction material. Since the concrete is required to develop a minimum compressive strength of 5,000 psi (35 MPa), it will generally exceed 8,000 psi (56 MPa) at 28 days. With its enhanced performance characteristics of high early strength and increased durability, HES concrete would be applicable for a variety of structural members, full-depth pavement patches, or new pavement construction and overlays. It should be extremely useful in situations where the speed of construction is important but not critical, even though the materials may be relatively more expensive. The following are potential applications for HES concrete:

New Pavement
Pavement Overlay
Full-Depth Pavement Patch
Full Bridge Deck Replacement
New Bridge Deck
Bridge Deck Overlay
Prestressed Bridge Girders
Precast Elements
Prestressed Piles/Columns/Piers

2

Objective and Scope

The objective of this investigation was to develop and analyze basic data on the mechanical properties of high early strength (HES) concrete for highway applications. The concrete was produced using *only locally available conventional constituent materials and normal production and curing procedures.* So that the research results would be applicable to different regions, the study included four different types of coarse aggregate and three kinds of sand, as summarized in Table 2.1. The chosen materials are representative aggregates from a wide geographical area.

Table 2.1 Types of coarse and fine aggregates

Type	Symbol	Source
Marine Marl	MM	Castle Hayne, North Carolina
Crushed Granite	CG	Garner, North Carolina
Dense Crushed Limestone	DL	West Fork, Arkansas
Washed Rounded Gravel	RG	Memphis, Tennessee
Sand		Lillington, North Carolina
Sand		Memphis, Tennessee
Sand		Van Buren, Arkansas

Both laboratory experiments and field studies were conducted. The laboratory experiments included eight different types of tests: compression, tension (both flexure and splitting), freezing-thawing, shrinkage, rapid chloride permeability, AC impedance, concrete-to-concrete bond, and concrete-to-steel bond. The field studies were carried out at five different sites in New York, North Carolina, Illinois, Arkansas, and Nebraska, which represented a wide range of environmental conditions.

The studies using crushed granite (CG), marine marl (MM), and washed rounded gravel (RG) were conducted at North Carolina State University; the studies using dense crushed limestone (DL) were conducted at the University of Arkansas. Lillington sand was used for the concretes made with CG or MM, but for the concrete made with RG, Memphis sand was used. Van Buren sand from the Arkansas River was used for the concrete made with DL.

3

Characterization of Constituent Materials

3.1 Cements

Type III cement was used for the production of high early strength (HES) concrete. The cement used at North Carolina State University (NCSU) was of low alkali content and met the requirements of ASTM C 150 specifications. It was supplied by Blue Circle Cement, Inc., from its plant in Harleyville, South Carolina. The cement used at Arkansas was supplied by the same manufacturer from its plant in Tulsa, Oklahoma.

The results of physical and chemical analyses of the two kinds of Type III cement are summarized in Table 3.1.

3.2 Coarse Aggregates

Four different types of coarse aggregate were used in this test program. They were chosen as representative aggregates from a wide geographical area. Crushed granite (CG) is a strong, durable aggregate locally available in North Carolina. It was supplied by Martin Marietta Co. from its quarry in Garner, North Carolina. Marine marl (MM) is a weaker and more absorptive aggregate available in the coastal area of North Carolina. It was also supplied by Martin Marietta Co. from its quarry in Castle Hayne, North Carolina. Washed rounded gravel (RG) was provided by Memphis Stone and Gravel Co., from its Pit 558 in Shelby County, Tennessee. Dense crushed limestone (DL) was supplied by McClinton-Anchor from its West Fork quarry just outside of Fayetteville, Arkansas.

The coarse aggregates used at NCSU met ASTM C 33 size #57 specifications, with most of the material passing the 1-in. (25-mm) sieve. The CG was a hard, angular aggregate of low absorption (0.6%). The MM was a cubical to subangular, relatively porous, and highly absorptive (typically more than 4.5%) shell limestone. The RG, drawn from a river, was primarily silicious and contained some crushed faces, but most of them were worn. The absorption was moderate (just under 3%), and hard chert particles were present.

Table 3.1 Results of physical and chemical analyses of Type III cement compared with ASTM C 150

	ASTM C 150 Type III	Type III* NCSU	Type III+ Arkansas
Fineness			
Specific surface (Blaine)	—	4,575 cm^2/g	5,590 cm^2/g
Soundness			
Autoclave expansion	0.80%	-0.03%	0.02%
Time of setting (Gillmore)			
Initial	1 hr	3 hr	1 hr 48 min
Final	10 hr	6 hr	2 hr 43 min
Water required			
1:2.75 mortar cubes	—	48.5%	—
Air temperature	—	73°F	—
Relative humidity	—	70%	—
Compressive strength (psi), 2 in. mortar cubes			
1 day	1,800	3,400	3,717
3 days	3,500	4,450	5,258
7 days	—	—	5,725
Silicon dioxide (SiO_2), %	—	20.4	20.1
Aluminum oxide (Al_2O_3), %	—	5.1	5.6
Ferric oxide (Fe_2O_3), %	—	3.8	2.1
Calcium oxide (CaO), %	—	65.2	64.1
Magnesium oxide (MgO), %	6.0	1.0	2.4
Sulfur trioxide (SO_3), %	3.5	2.9	3.7
Loss on ignition, %	3.0	1.2	1.1
Sodium oxide (Na_2O),%	—	—	0.2
Potassium oxide (K_2O), %	—	—	0.7
Total equivalent alkali content, %	0.60	0.18	0.66
Tricalcium silicate, %	—	62.6	58.6
Dicalcium silicate, %	—	11.2	13.3
Tricalcium aluminate, %	15	7.2	11.4
Tetracalcium aluminoferrite, %	—	11.5	6.4
Insoluble residue, %	0.75	0.03	0.15

Note: 1 MPa = 145 psi
* Tests performed by the Materials and Tests Unit of North Carolina DOT
+ Tests performed by the Materials Division of Arkansas DOT

The maximum size of DL used at Arkansas was slightly smaller and met ASTM C 33 size #67 specifications.

Mineralogically, the CG consisted of approximately 35% quartz, 30% potassium feldspar, 25% sodium-rich plagioclase feldspar, and 10% biotite. The MM was a sandy fossiliferous limestone with about 60% calcite, 35% quartz, and 5% other oxide and hydroxide minerals. The RG consisted of 25% quartz, 10% quartzite, 60% chert, and 5% sandstone. The DL contained about 97% limestone and 3% clay minerals. It should be noted that the RG contained a large amounts of chert, which could be a cause for alkali-silica reaction.

Physical anlyses of the coarse aggregates were performed according to ASTM C 33, and the results are shown in Table 3.2.

Table 3.2 Properties of coarse aggregates

	CG	MM	RG	DL
Specific gravity (SSD)	2.64	2.48	2.55	2.72
% absorption	0.6	6.1	2.8	0.69
DRUW (pcf)	93.6	78.4	94.8	99.0
Fineness modulus	6.95	6.92	6.99	6.43
% Passing				
1 in.	100	98	95	100
3/4 in.	90	85	72	100
1/2 in.	31	43	56	82
3/8 in.	13	19	26	48
#4	2	4	1	6
#8	0	0	2	3
L.A. abrasion, %				
Grading A	—	—	17.6	—
Grading B	39.6	43.7	—	24
Sodium sulfate soundness, %	1.3	9.6	2.8	3
Less than 200 by washing, %	0.6	0.4	—	—

3.3 Fine Aggregates

Three different kinds of sand were used in this test program. The sand used with CG and MM was obtained from Lillington, North Carolina. The sand used with RG was from Memphis, Tennessee, and Arkansas River sand from Van Buren, Arkansas was used with DL.

The Lillington sand contained 75% quartz, 22% feldspar, and 3% epidote. The finer material (passing #10 sieve) of the Memphis sand consisted of 95% quartz, 4% opaque minerals (oxide and hydroxide minerals), and 1% other miscellaneous minerals; the coarser material (retained on #10 sieve) consisted of 20% chert, 30% sandstone and shale fragments, and 50% quartz. The finer material of the Van Buren sand consisted of 85% quartz, 4% chert, and 11% microcline and less than 1% rock fragments and heavy minerals; the coarser material consisted of 62% quartz, 16% chert, 11% microcline, and 5% rock fragments. The physical properties of the three kinds of sand are shown in Table 3.3.

Table 3.3 Properties of fine aggregates

	Lillington Sand	Memphis Sand	Van Buren Sand
Specific gravity (SSD)	2.57	2.62	2.62
% absorption	1.1	1.2	0.6
Fineness modulus	2.66	2.60	2.72
% passing			
#4	100	100	96
#8	97	93	88
#16	80	82	75
#30	47	55	55
#50	9	9	13
#100	1	1	0.4

3.4 Chemical Admixtures

Chemical admixtures used in the various concrete mixtures of this test program included an accelerator, two types of high-range water reducer (HRWR), an air entraining agent (AEA), and a retarder. Their brand names, suppliers, and respective reference specifications are identified in Table 3.4.

Table 3.4 Chemical admixtures used in the test program

Admixture	Brand Name	Supplier	Reference Specifications
Accelerator	DCI (Calcium Nitrite) - 30% Solution	W. R. Grace	ASTM C 494, Type C
HRWR	Melment 33% (Melamine Base)	Cormix	ASTM C 494, Type F
HRWR	PSI Super (Naphthalene Base)	Cormix	ASTM C 494, Type F
AEA	Daravair (Neut. Vin. Resin) - 17% Solids	W. R. Grace	ASTM C 260
Retarder	PSI 400R (Lignin Base)	Cormix	ASTM C 494, Type D

3.5 Other Admixtures

Latex was the only other admixture used in this investigation. To a limited extent, it was used in one test series of HES concrete. The latex was a styrene butadiene, Mod A (Lot MM 90102303), supplied by Dow Chemical.

4

Mixture Proportions

In the early stage of this investigation, extensive development work involving a total of 360 trial batches of concrete was conducted to determine appropriate mixture proportions for the various types of high performance concrete. A detailed discussion of this development work can be found volume 2 of this report series, *Production of High Performance Concrete*. For HES concrete, the development work included 81 trial batches using four different kinds of coarse aggregate.

Proportioning of the concrete mixtures was based on the methods in ACI 211, *Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete*. Selections of water/cement ratio (W/C), workability, and air content requirements were made first and these constraints were incorporated in accordance with the ACI 211 guidelines, as was selection of aggregate quantities.

A nominal maximum size (NMSA) of 1 in. (25 mm) (ASTM C 33, size #57) for the coarse aggregate was selected as being the most appropriate for a variety of applications. This aggregate size could be used for many structural members or pavements, although larger aggregates might be better for paving mixes in some locations.

Similarly, although the quantity of coarse aggregate (volume of coarse aggregate per unit volume of concrete) could have been increased by about 10% for paving applications, the quantity initially selected was at or near the recommended value in ACI 211. These values were adjusted slightly in subsequent trial batches. The objective of selecting a less coarse mixture was to provide a more general-purpose mixture that would be easier to use in small hand-placed and hand-finished applications.

Following evaluation of the results of the trial batches, the mixture proportions for HES concrete with four different types of aggregate were selected. They are summarized in Table 4.1.

Table 4.1 Mixture proportions of HES concrete with four different aggregate types

Aggregate type: Source of sand:	CG Lillington	MM Lillington	RG Memphis	DL Van Buren
Cement (Type III), pcy	870	870	870	870
Coarse aggregate, pcy	1,720	1,570	1,650	1,680
Sand, pcy	960	980	900	1,030
HRWR (Naphthalene Based), oz/cwt	26	26	26	16
Calcium nitrite (DCI), gcy	4.0	4.0	4.0	4.0
AEA, oz/cwt	9	1.0	1.0	4.0
Water, pcy	280	280	300	300
W/C	0.32	0.32	0.34	0.34
Slump, in.	1.0	6.75	7.0	3.5
Air, %	5.3	5.6	6.6	5.4
Strength at 1 day, psi	5,410	5,610	5,690	5,300
Concrete temperature at placement, ^{o}F	80	73	84	78

Note: 1 MPa = 145 psi

5

Mixing and Curing Procedures

5.1 Mixing Procedures

Concretes made with crushed granite (CG), marine marl (MM), or washed rounded gravel (RG) were produced in the Concrete Materials Laboratory at North Carolina State University (NCSU) using a tilt drum mixer with a rated capacity of 3.5 ft^3 (0.1 m^3). Concretes using dense crushed limestone were produced in an identical mixer in the Concrete Laboratory at the University of Arkansas. The normal laboratory mixing and batching procedures (ASTM C 192) were modified slightly to represent typical concrete dry-batch plant operations more closely. Whether the concrete was produced at NCSU or Arkansas, the same general mixing procedures were followed.

HES concrete used Type III portland cement and calcium nitrite (DCI) as an accelerator. The mixing procedure followed these steps:

1. Butter the mixer with a representative sample of mortar composed of approximately 3 lb (1.36 kg) cement, 6 lb (2.73 kg) sand, and 2 lb (0.91 kg) water. Turn on the mixer to coat the interior completely. (At Arkansas, only water was used to butter the mixer.) Empty the mixer and drain it for 1 minute.

2. Charge the mixer successively with approximately 25% of the coarse aggregate, 100% of the sand, 50% of the water and 100% of the air entraining agent added with the sand. Mix for 1 minute to generate air bubbles. (At Arkansas, the mixer was charged with 50% of the coarse aggregate, 67% of the sand, and 67% of the water.)

3. Stop the mixer and add remaining coarse aggregate and water. Mix for 1 minute (if needed) to equalize the temperature of the materials. Otherwise, mix for 10 seconds and then add the entire amount of cement. Record the time as the beginning of the total mixing time. After mixing for 1 minute, stop the mixer; add the high-range water reducer and continue mixing for 5 minutes. During all mixing, cover the mixer with a lid to minimize evaporation.

4. Mix continuously for 20 minutes to simulate travel time for a ready-mix truck. Then stop the mixer and add calcium nitrite (DCI). Mix for an additional 5 minutes. (Total mixing time beginning with the addition of cement is approximately 30 minutes.)

5. Discharge the concrete into a wheelbarrow; measure unit weight, air content, slump, and temperature; fabricate test specimens.

5.2 Curing Procedures

Curing procedures were developed to simulate more closely the conditions in the field, and they differed considerably from the normal laboratory curing procedure (ASTM C 192). The procedures differed for different categories of high performance concrete. For HES concrete, the demand for early strength gain was not as critical as for VES concrete; thus it was not necessary to use insulation.

Cylinders of HES concrete were cast in 4 x 8-in. (100 x 200-mm) plastic molds and beam specimens were cast in steel forms. These specimens were maintained at 60° to 80°F (15.6° to 26.7°C) and were protected from evaporation until an age of 20 to 24 hours. Several specimens were then removed from molds and tested immediately; others were placed in sealed plastic bags for testing at later ages.

6

Laboratory Experiments

The laboratory investigation consisted of tests for both fresh or plastic concrete and hardened concrete. The plastic concrete tests included tests for slump, air content, etc; the results of these tests are presented in volume 2 of this report series, *Production of High Performance Concrete*. The hardened concrete tests included compression tests for strength and modulus of elasticity, tension tests for tensile strength and flexural modulus, freeze-thaw tests for durability factor, shrinkage tests, rapid chloride permeability tests, tests for AC impedance, tests for concrete-to-concrete bond, and tests for bond between concrete and steel reinforcement. The testing program for the mechanical properties of hardened concrete is outlined in Tables 6.1 through 6.4*.

6.1 Compression Tests

The compression tests were conducted on 4 x 8 in. (100 x 200 mm) cylinders at different ages to obtain stress-strain, strength-time, and modulus-time relationships for HES concrete with four different types of coarse aggregates (MM, CG, DL, and RG). A limited number of compression tests were also conducted on 6 x 12 in. (150 x 300 mm.) cylinders to investigate the size effect.

6.1.1 Test Setup and Procedure

The tests were conducted in a 2,000 kip (8,900 kN) compression testing machine with a hydraulic feedback system and an MTS 436 controller unit. The machine was capable of both load and displacement control modes; the tests for strength and modulus of elasticity were done using the load control option.

The tests were conducted according to AASHTO T-22-86 and ASTM C 39, with minor modifications. The modifications were the use of an unbonded capping system, such as steel caps lined with neoprene pads, and the use of a deformation measuring fixture that included

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*Testing for the fatigue properties of plain concrete (Group 4) was not conducted due to the change of the scope of the project.

Table 6.1 Test program for compressive strength and modulus of elasticity — Group 1

Category of HPC	Aggregate Type	No. of 4 x 8-in. Cylinders Tested				No. of 6 x 12-in. Cylinders Tested
		1 Days	3 Days	7 Days	28 Days	1 Day
HES	MM	3	3	3	3	2
	CG	3	3	3	3	2
	DL	3	3	3	3	2
	RG	3	3	3	3	2
HES (L)*	MM	3	3	3	3	2
HES (std)*	RG	3	3	3	3	2

* L = latex, std = standard cure

Table 6.2 Test program for modulus of rupture, tensile strain capacity, and split tensile strength — Group 2

Category of HPC	Aggregate Type	No. of Rupture Specimens Tested			No. of Split Tensile Specimens Tested	No. of 4 x 8-in. "Control" Cylinders Tested
		1 Day	7 Days	28 Days	1 Day	1 Day
HES	MM	2	2	2	2	2
	CG	2	2	2	2	2
	DL	2	2	2	2	2
	RG	2	2	2	2	2
HES (L)*	MM	2	2	2	2	2
HES (std)*	RG	2	2	2	2	2

* L = latex, std = standard cure

Table 6.3 Test program for frost durability, shrinkage, and chloride permeability — Group 3

Category of HPC	Aggregate Type	No. of Freezing - Thawing Specimens Tested at 14 Days	No. of Shrinkage Specimens Tested from Design Age to 90 Days	No. of Chloride Permeability Specimens Tested at 14 Days	No. of 4 x 8-in. "Control" Cylinders Tested at 1 Day
HES	MM	—	3	2	2
	CG	3	3	2	2
	DL	3	3	2	2
	RG	—	3	2	2
HES (L)*	MM	—	—	2	2
HES (std)*	RG	3	3	2	2

* L = latex, std = standard cure

Table 6.4 Test program for bond strength — Group 5

Category of HPC	Aggregate Type	No. of Specimens of Concrete-to-Concrete Bond Test at 1 Day	No. of Specimens of Concrete-to-Steel Bond Test at 1 Day	No. of 4 x 8-in "Control" Cylinders Tested at 1 Day
HES	MM	2	—	—
	CG	2	2	2
	DL	—	—	—
	RG	—	—	—
HES (L)*	MM	—	—	2
HES (std)*	RG	—	—	—
NCDOT*	CG	2	—	—

* L = latex, std = standard cure, NCDOT = standard NCDOT concrete as control

linear voltage differential transducers (LVDTs). The two LVDTs used in the compressive tests were Lucas Schaevitz MHR 050, with a sensitivity of 2.500 mV/V/0.001 in. The voltage was converted by a National Instruments AT-MIO-16 12-bit analog-to-digital converter.

An aluminum mounting jig was built to facilitate the mounting of the LVDTs on the test cylinder. The jig ensured that the LVDTs were placed 180 degrees apart and in the central 4 in. (100 mm) of the test cylinders. The device for mounting the transducers on the 4 x 8-in. (100 x 200-mm) cylinders is shown in Figure A.1. First the specimens were placed in the device and the bottom of the cylinder was placed so as to align with the alignment mark. A line of super-glue was applied at each of the four locations where the transducers were to be attached. The metallic contact area of the LVDT holder was sprayed with a zip kicker (a product for speeding up the reaction between the metal and the concrete surface with super-glue), and the LVDT holders together with LVDTs were attached with the wires of the LVDT protruding toward the bottom of the cylinder. After about a minute, the cylinder was removed from the transducer mounting device. To ensure a good bond between the LVDT holders and the concrete surface, additional super-glue was applied all around the contact surface and sprayed with the zip kicker. The unbonded caps were put on the cylinders and the specimen was placed in the compression testing machine.

The LVDTs used to measure the axial deformation of the cylinders had a maximum range of ± 0.05 in. (1.27 mm) and the gage length for the axial deformation measurements was 4 in. (100 mm). The core of the LVDTs could be moved up or down by a specially designed screw-thread mechanism (which was machined to become part of the core of the LVDT) so as to adjust the output voltage to zero or near zero.

The test specimen with the two mounted LVDTs was placed inside a protective steel jacket that rested between the two platens of the compression testing machine (Figure A.2). The steel jacket prevented damage to the transducers from the brittle failure of the specimens at the maximum load.

The test cylinders were loaded and unloaded up to a load of 5,000 lbs (22.2 kN) at least twice before the cylinders were loaded to failure. This initial loading and unloading was done for seating purposes and to properly zero out the LVDTs. After the initial loading and unloading, the cylinders were loaded to failure. For 4 x 8-in. (100 x 200-mm) cylinders, a loading rate of 26,500 lbs/min (118 kN/min) was used; for the 6 x 12-in. (150 x 300-mm) cylinders, a loading rate of 59,400 lbs/min (264 kN/min) was used. The loading rates were within the range of 20 to 50 psi/sec (0.14 to 0.34 MPa/sec) specified in AASHTO T-22-86 and ASTM C 39 revised.

The data acquisition system used was an OPTIM system (Megadac 100) capable of recording up to 40 channels of output. The load output and the output from the two displacement transducers were recorded using LabWindows software in conjunction with the OPTIM data acquisition system. A view of the compression test setup is shown in Figure A.3.

6.1.2 Specimen Preparation

The specimens were prepared according to ASTM C 192. Specimens used specifically for determining the compressive strength and modulus of elasticity were 4 x 8-in. (100 x 200-mm) cylinders and cast in plastic molds. Companion 6 x 12-in. (150 x 300-mm) cylinders were also cast in plastic molds to investigate the size effect.

After casting, the specimens inside the molds were maintained at the normal laboratory temperature of 60° to 80°F and were protected from moisture evaporation by a plastic sheet cover for 20 to 24 hours. Then they were stripped of their molds and either tested immediately or placed in sealed plastic bags for testing at later ages. The specimens were air dried for 10 to 15 minutes before the two LVDTs were mounted on the sides of the specimens.

6.1.3 Test Results and Discussions

There are three categories of test results. They include stress-strain relationships, strength-time and modulus-time relationships, and strength comparisons of 4 x 8-in. (100 x 200-mm) cylinders with 6 x 12-in. (150 x 300-mm) cylinders.

6.1.3.1 Stress-Strain Relationships

The stress-strain curves were obtained from the load-deformation curves by dividing the load by the nominal area of the cylinders and the axial deformation by the gage length. Figure 6.1 shows typical stress-strain curves of HES concrete at the design age of 1 day with different types of coarse aggregates. From these figures, it appears that the stress-strain curve for HES concrete with latex and marine marl (MML) is relatively more nonlinear than HES concretes with MM, CG, RG, and DL. The strain capacity corresponding to the maximum strength is about 3,000 microstrains for HES concrete with MML compared with about 1,500 microstrains for HES concretes with MM, CG, RG, and DL.

The effect of coarse aggregate on the stress-strain relationship at 28 days is shown in Figure 6.2. It can be seen that HES concretes with MM and CG exhibit softer response in the initial portion of the curve than HES concrete with RG. This translates into a lower modulus of elasticity for concrete with MM and CG. The compressive strain capacity at 28 days ranges from 1,500 to 2,000 microstrains (Figure 6.2).

The effect of age on the stress-strain curve for HES concrete with one type of coarse aggregate (MM) is shown in Figure 6.3. The initial portion of the stress-strain curve becomes more linear as the concrete matures, and the effect on the strain capacity is negligible. Note that the stiffening of the behavior in the initial portion of the stress-strain curve with time results in an increase in the modulus of elasticity with time.

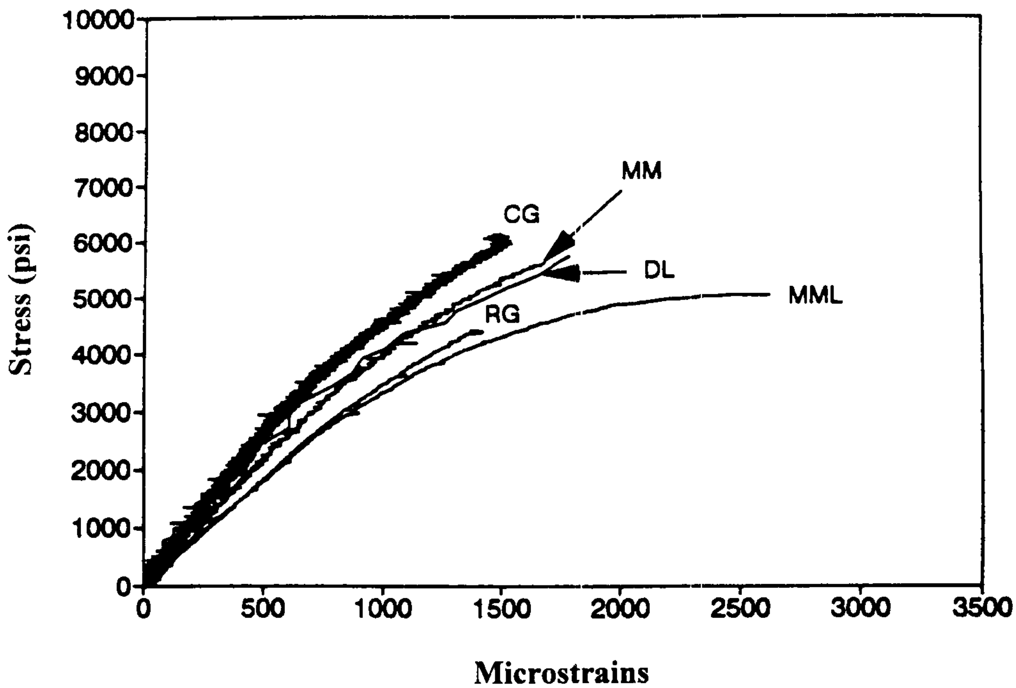

Figure 6.1 Stress-strain relationship of HES concrete at design age of 1 day

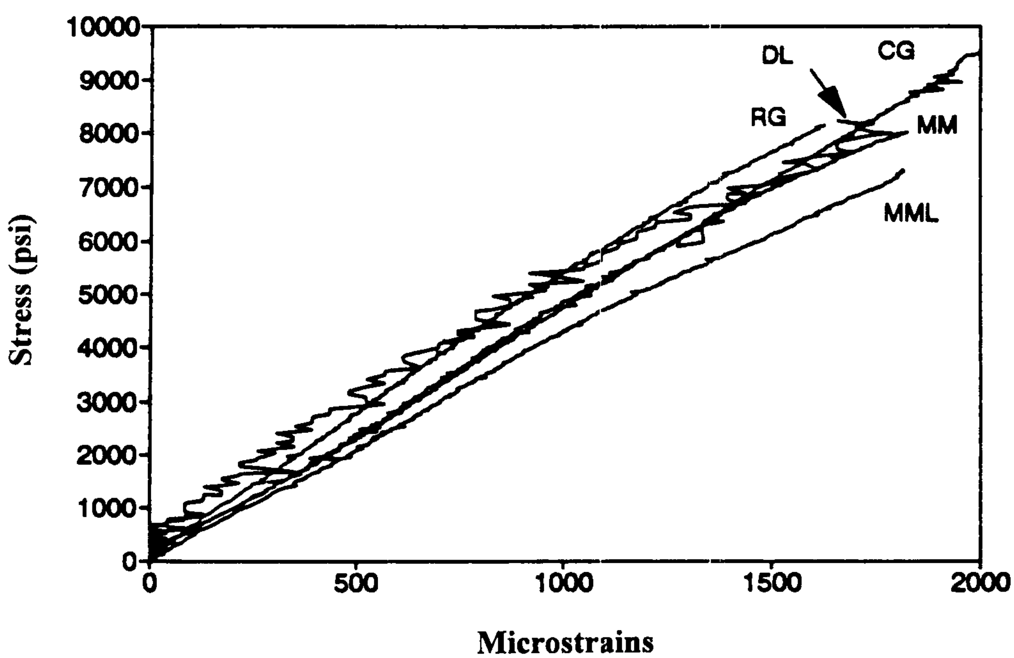

Figure 6.2 Effect of coarse aggregate on stress-strain relationship at 28 days

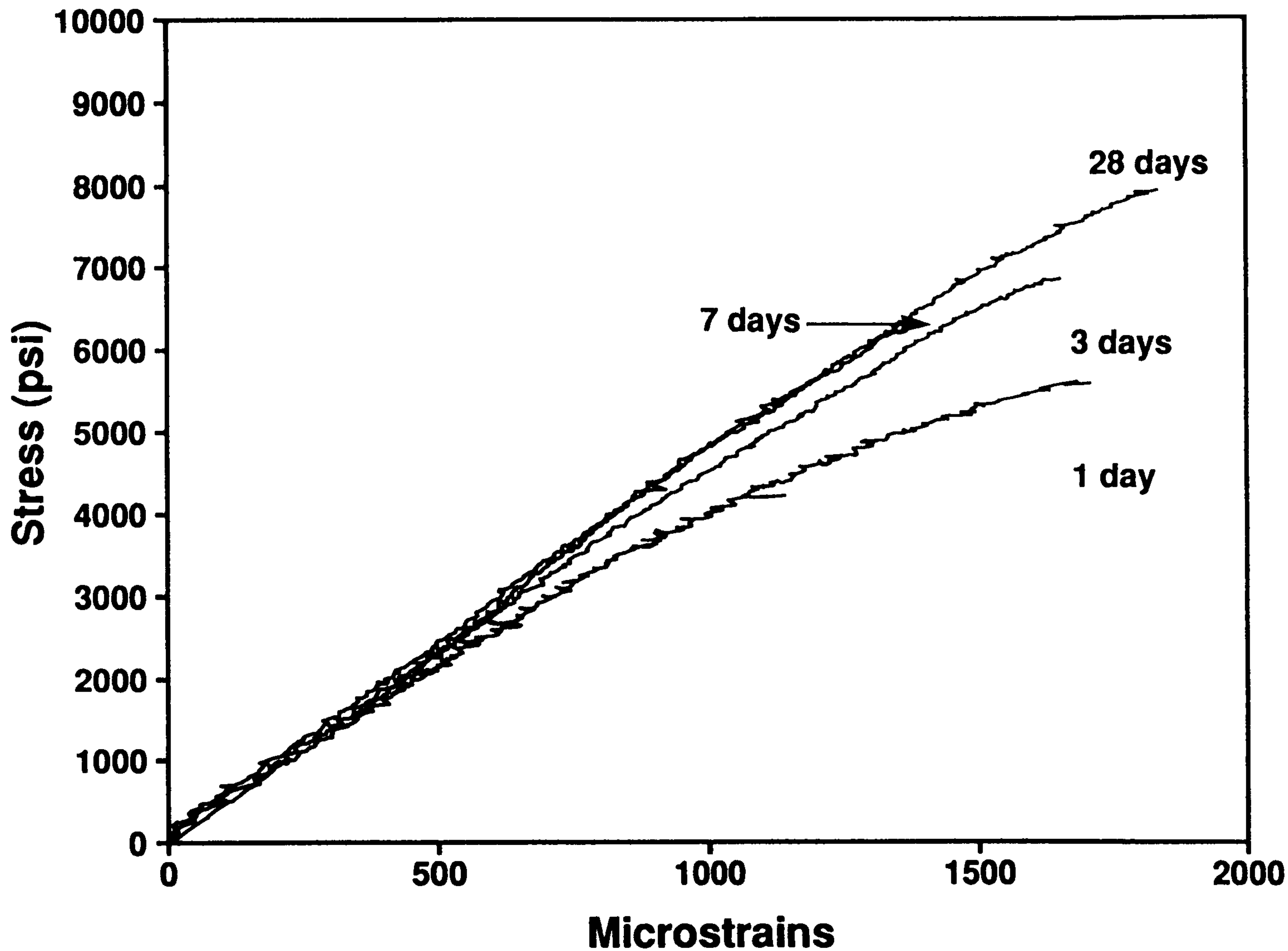

Figure 6.3 Effect of age on stress-strain curve of HES concrete with MM

6.1.3.2 Strength-Time and Modulus-Time Relationships

The strength and modulus of elasticity test results for HES concrete are summarized in Table 6.5. The values shown in the table are the averages of three replicate specimens. The strength-time relationships for the different types of coarse aggregate in a nondimensional form are shown in Figure 6.4. Each point on the curve is based on an average of three replicate specimens. It can be seen that the strength gain is relatively faster for concrete with MM and DL aggregates than concretes with CG and RG aggregates. The effect of the coarse aggregate type on the strength seems to become less significant after 14 days. The comparison of the observed strength-time relationships of HES concretes with the prediction per ACI Committee 209 (1993a) is also shown in Figure 6.4. The comparison shows that the strength gain is much faster in the first 15 days for HES concrete than predicted by ACI Committee 209. Note that the equation of ACI Committee 209 was developed from a large body of experimental data for concretes with strengths up to 6,000 psi (42 MPa) at 28 days.

The modulus-time relationships for the different types of coarse aggregate are shown in Figure 6.5. Each point on the curve is an average of three replicate specimens. The results indicate lower modulus for the softer coarse aggregate such as MM; however, the rate of increase in the modulus with time is comparable to the other types of coarse aggregates.

The predictions by the various suggested equations for the modulus of elasticity by ACI 318 (1993c), ACI 363 (1993d), Ahmad and Shah (1985), and Cook (1989) were compared with the observed modulus of elasticity of HES concrete at the design age of 1 day (Figure 6.6). The equation as suggested by ACI 318 uses a premultiplier of 57,000 to the square root of the compressive strength in psi units. It can be seen that the ACI 318 equation provides a better prediction than those of ACI 363, Ahmad and Shah and Cook. This is because the ACI 318 equation was developed for lowe-strength concretes up to 6,000 psi (42 MPa) at 28 days, whereas the other equations were developed primarily for concretes with strengths exceeding 10,000 psi (70 MPa) at 28 days. Note that the modulus of elasticity does not change significantly after the design age of 24 hours (Figure 6.5).

6.1.3.3 Strength Comparisons of 4 x 8-in. and 6 x 12-in. Cylinders

A summary of the strength comparisons between the 4 x 8-in. (100 x 200-mm) cylinders and the 6 x 12-in. (150 x 300-mm) cylinders is presented in Table 6.6. The results indicate that the ratio of the 6 x 12-in. (150 x 300-mm) cylinder strengths to the 4 x 8-in. (100 x200-mm) cylinder strengths varies with the type of coarse aggregate used. The ratio varies from 0.90 to 1.05. Such ratios reported in the literature for concretes with 28-day strength in the range of 5,000 psi (35 MPa) to 12,000 psi (84 MPa) vary from 0.90 to 0.95 (Carrasquillo et al. 1981, Leming 1988, Moreno 1990).

Table 6.5 Summary of test results for compressive strength and modulus of elasticity at different test ages

Age (days)	MM	CG	DL	RG	MML	RG (std)
			Compressive Strength (psi)			
1[1]	5,610	5,140	5,660	5,820	4,230	5,690
1[2]	6,370	5,380	5,590	4,300	4,790	5,120
1[3]	6,050	6,000	6,050	4,620	4,920	5,260
3	6,850	7,260	—	6,050	5,480	6,540
7	7,360	8,290	7,690	6,850	6,030	7,110
28	7,990	9,540	8,170	8,130	7,180	7,760
			Modulus of Elasticity (10^6 psi)			
1[1]	4.95	3.48	—	3.40	2.85	4.38
1[3]	4.05	5.00	5.23	4.05	3.55	3.65
3	4.55	6.25	—	3.75	3.90	5.00
7	4.85	5.00	5.03	5.55	4.05	5.35
28	5.30	4.80	5.41	5.50	4.25	5.65

[1] 4 x 8-in. companion cylinders of flexural strength testing.
[2] 6 x 12-in. companion cylinders of compressive strength testing.
[3] 4 x 8-in. cylinders of compressive strength testing.

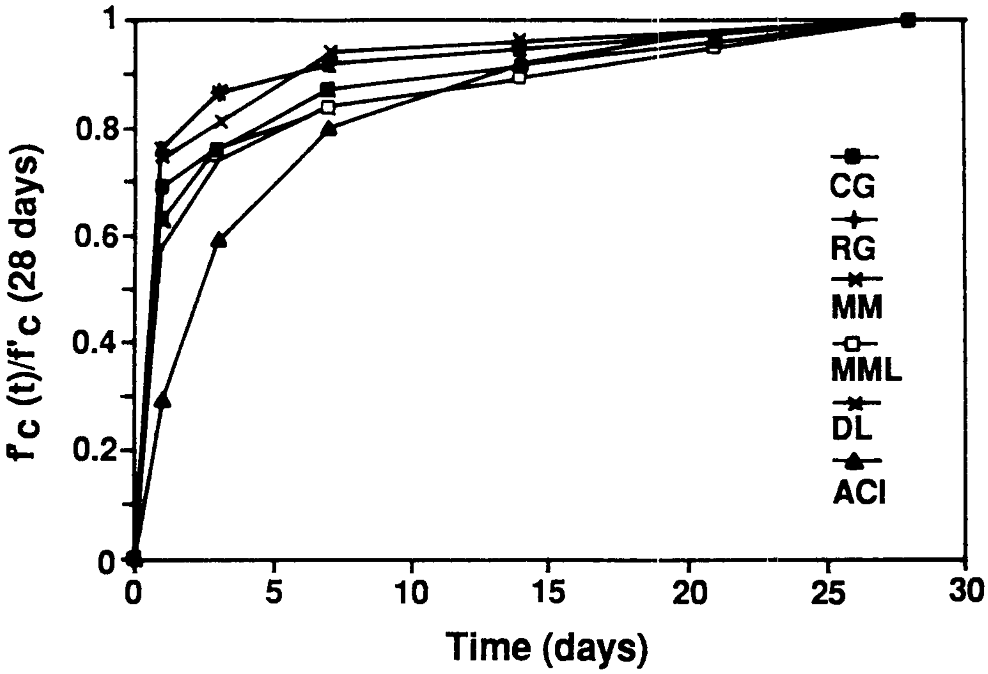

Figure 6.4 Variation of compressive strength with time for HES concrete

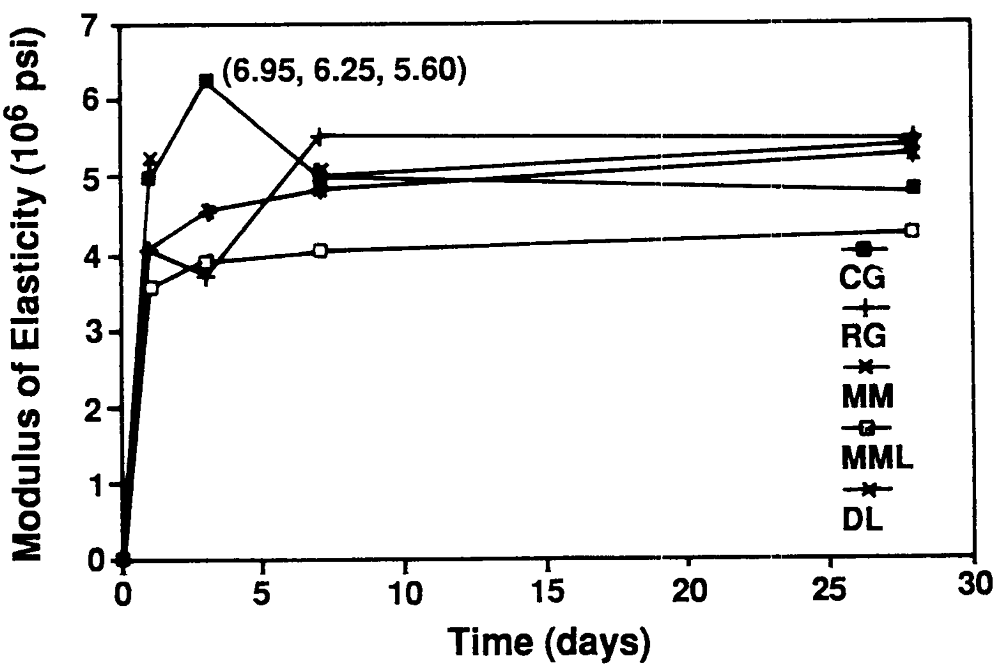

Figure 6.5 Variation of modulus of elasticity with time for HES concrete

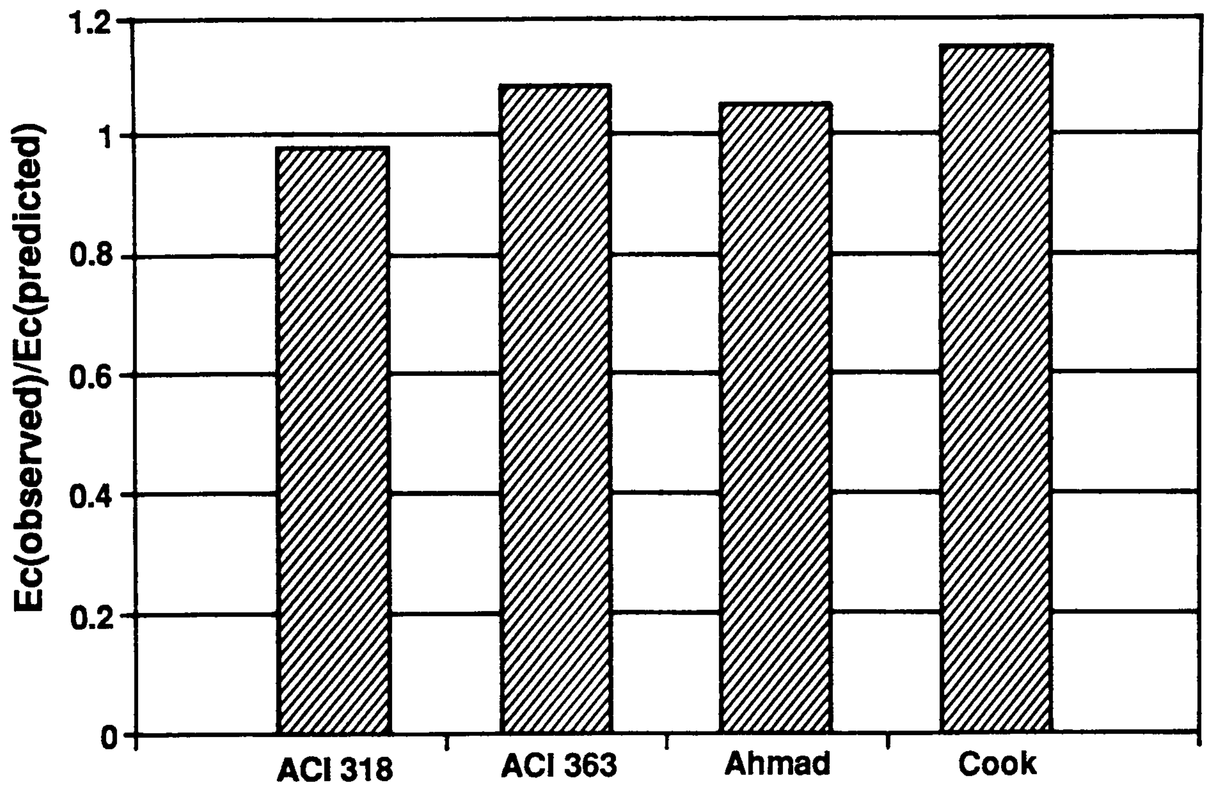

Figure 6.6 Comparison of observed vs. predicted modulus of elasticity of HES concrete

Table 6.6 Summary of test results for 4 x 8-in. and 6 x 12-in. cylinder strengths for HES concrete

Specimen Size	No. of Specimens	Test Age (days)	Compressive Strength (psi)				
			MM	CG	DL	RG	MML
A: 4 x 8-in.	3	1	6,050	6,000	6,050	4,620	4,920
B: 6 x 12-in.	2	1	6,370	5,380	5,590	4,300	4,790
B/A			1.05	0.90	0.92	0.93	0.97

6.2. Tension Tests

Two types of tensile strength tests were conducted: split cylinder tests and flexural tests. The split cylinder tests were conducted on 4 x 8-in. (100 x 200-mm) cylinders, and the flexural tests were conducted on 4 x 4 x 17.5-in. (100 x 100x 437.5-mm) beams.

6.2.1 Test Set-Up and Procedure

6.2.1.1 Split Cylinder Tests

The split cylinder tests were conducted according to ASTM C 496. The cylinders were loaded at a rate of 7,500 lbs/min (33.4 kN/min) until failure. This rate of loading was within the ASTM specified range of 100 to 200 psi/min (0.69 to 1.38 MPa/min). The split cylinder tests were conducted on a 300 kip (1,335 kN) compression testing machine.

6.2.1.2 Flexural Tests

The flexural tests were conducted on a universal testing machine with a capacity of 120 kips (534 kN). The machine was equipped with a SATEC System Inc. M120BTE automated control system for programming the loading rate. The flexural tests were conducted in accordance with AASHTO T 97-86 and ASTM C 78, with some modifications. The modifications were necessary to incorporate the capability of monitoring the tensile strain capacity and the load-deflection response of the test specimen during testing. To measure the tensile strain and the midspan deflection of the test specimen, a mounting frame (fixture) was designed and fabricated. The mounting frame is capable of holding four LVDTs (two on each side of the beam) and one for measuring the mid-span deflection. To prevent damage to the transducers, a No. 2 smooth reinforcing bar was placed along the centroidal axis of the beam. This prevented the sudden collapse of the specimen upon reaching the maximum load and protected all the transducers.

Several preliminary tests indicated that the use of a No. 2 smooth reinforcing bar along the centroidal axis did not have any detectable effect on the strength and behavior of the test specimens. The test specimens were first placed in a transducer mounting device to facilitate the mounting of the four transducers to monitor the tensile and compressive strains during the flexural test. The device for mounting the transducers is shown in Figure A.4.

Preliminary testing revealed that the combination of slight imperfections in the steel molds for the beams and the rigid supports introduced a torsional effect that changed the mode of failure of the test specimen. A special beam support unit that could accommodate these imperfections was then designed and fabricated. In the beam support unit, both of the supports are restrained against motion along the centroidal axis of the beam; however, in the transverse direction, one of the supports is allowed to rotate (Figure A.5).

The mounting frame on the flexural test beams for monitoring the midspan deflections is shown in Figure A.6. The beams were tested in third-point loading over a clear span of 12 in. (300 mm) according to AASHTO T 97-86 and ASTM C 78. The loading arrangement for the flexural testing is shown in Figure A.7.

The beams were tested at different ages, and companion 4 x 8-in. (100 x 200-mm) cylinders were tested at the same time. The test beams were loaded and unloaded up to a load of 500 lbs (2.3 kN). This process was done twice for seating and zeroing the LVDTs. After the initial loading and unloading process, the test beams were loaded to failure. A loading rate of 800 lbs/min (3.6 kN/min) was used.

The four LVDTs used to measure displacements in the flexural tests were Trans-Tek, Inc. #0270-0000, with a sensitivity of 3.189 VAC/ Inch/ Volt Input. A gage length of 4 in. (100 mm) was used for monitoring the compressive and tensile deformation near the extreme fibers. The voltage output from the LVDTs was converted by an OPTIM data acquisition system (Megadec 100). This system was used to eliminate electronic noise in order to record accurately the small displacements encountered. An aluminum jig was used for mounting the four LVDTs in the middle third of the beam and 5/8 in. (15.6 mm.) from the top fiber and bottom fiber on the front and back of the beam. A view of the test setup is shown in Figure A.8.

6.2.2 Specimen Preparation

Specimens used for flexural tests were 4 x 4 x 17.5-in. (100 x 100 x 437.5-mm) beams, cast in steel molds with a No. 2 smooth bar placed along the centroidal axis. The smooth No. 2 bar was placed in the specimen to keep it from collapsing at failure in order to avoid damaging LVDTs. The concrete was placed into molds and vibrated internally with a needle vibrator. The finish was completed with a magnesium float. Companion 4 x 8-in (100 x 200-mm) cylinders were cast in plastic molds. After casting, the specimens inside the molds were maintained at 60° to 80°F and protected from moisture evaporation by a plastic sheet cover for 20 to 24 hours. Then they were stripped of their molds and either tested immediately or placed in sealed plastic bags for testing at later ages. Since the tests involved the measurements of tensile strains and midspan deflections, transducers were mounted on the test specimens. The specimens were air-dried for 10 to 15 minutes before the LVDTs were mounted on the two sides of the specimens. Figure A.4 shows the device for mounting the transducers on the specimen for flexural testing.

6.2.3 Test Results and Discussions

6.2.3.1 Split Cylinder Tests

The results of the split cylinder tests are shown in Table 6.7. The values shown in the table are the averages of two replicate specimens. The ratio of the observed to the predicted split cylinder strength for different types of coarse aggregates used for the production of HES concrete is shown in Figure 6.7. The equation suggested by ACI Code 318 (1993c) uses 6.7 as a premultiplier to the square root of the compressive strength in psi units. From this figure it appears that the split cylinder strength of HES concrete at the design age of 24 hours can be predicted with the empirical equations of ACI Code 318 since the equation was developed for lower-strength concretes up to 6,000 psi (42 MPa) at 28 days. Although the equation suggested by Ahmad and Shah (1985) was primarily for concretes with higher strengths (f_c' greater than 6,000 psi or 42 MPa) at 28 days, its prediction is relatively good for HES concrete at the design age of 24 hours. The prediction by the equation recommended by ACI Committee 363 (1993d) is least satisfactory, since the equation was developed for concretes with strengths well over 10,000 psi (70 Mpa) at 28 days.

6.2.3.2 Flexural Tests

The test results for the flexural modulus of HES concrete are presented in Table 6.7. The comparison of the experimental results with some of the empirical equations is shown in Figure 6.8 [ACI Committee 318 (1993c), ACI Committee 363 (1993d), Ahmad and Shah (1985)]. The equation suggested by ACI Code 318 (1993c) uses 7.5 as a premultiplier to the square root of the compressive strength in psi units. From Figure 6.8, it appears that the ACI 318-89 equation can be used to predict the modulus of rupture of HES concrete at the design age of 24 hours. The variation of the modulus of rupture with time is shown in Figure 6.9. This figure indicates that the modulus of rupture of HES concrete with CG is higher than for other types of coarse aggregates.

The load versus midspan deflection at design age (1 day for HES concrete) for all the coarse aggregate types is shown in Figure 6.10. The aggregate type does not seem to have an appreciable effect on the initial stiffness of the load versus midspan deflection response of the beams (Figure 6.10). The effect of age on load versus midspan deflection for one type of coarse aggregate is shown in Figure 6.11. This result indicates that with age the response tends to become stiffer in the initial portion.

The load vs. tensile strain for HES concrete for all the types of coarse aggregates at the design age of 1 day is shown in Figure 6.12. The results indicate that the tensile strain capacity at such an early age is not very sensitive to the type of the coarse aggregate and is essentially the same for all the types of coarse aggregates investigated. The load versus tensile strain at different ages (1, 3, 7, 28 days) for HES concrete with CG as coarse aggregate is shown in Figure 6.13. The results indicate that the tensile strain capacity remains essentially the same for a relatively young (1 day) concrete as for a 28-day-old concrete.

6.3 Freezing-Thawing Tests

6.3.1 Test Setup and Procedure

The freezing-thawing test was performed in accordance with ASTM C 666, procedure A, using a programmable freezing-thawing chamber as shown in Figure A.9. The chamber housed 12 rectangular aluminum containers 4 1/4 in. (108 mm) wide, 16 1/4 in. (413 mm) long, and 6 in. (152 mm) deep, surrounded by an antifreeze liquid that served as a heat exchange medium for the freezing-thawing cycle.

Table 6.7 **Summary of test results for modulus of rupture, tensile strain capacity, and split cylinder tensile strength for HES concrete**

Coarse Aggregate Type	Age (days)	Modulus of Rupture (psi)	Tensile Strain Capacity (microstrains)	Split Cylinder Strength (psi)	4 x 8-in. "Control" Cylinder Strength (psi)
MM	1	460	-258	440	5,610
	7	630	-175	—	—
	28	720	-163	—	—
CG	1	600	-146	380	5,140
	7	620	-164	—	—
	28	630	-188	—	—
DL	1	400	-165	590	5,660
	7	370	-178	—	—
	28	565	-180	—	—
RG	1	480	-115	540	5,820
	7	560	-163	—	—
	28	540	-125	—	—
MML	1	430	-165	370	4,230
	7	690	-170	—	—
	28	510	-175	—	—
RG (std)	1	430	-175	489	5,690
	7	624	-137	—	—
	28	580	-163	—	—

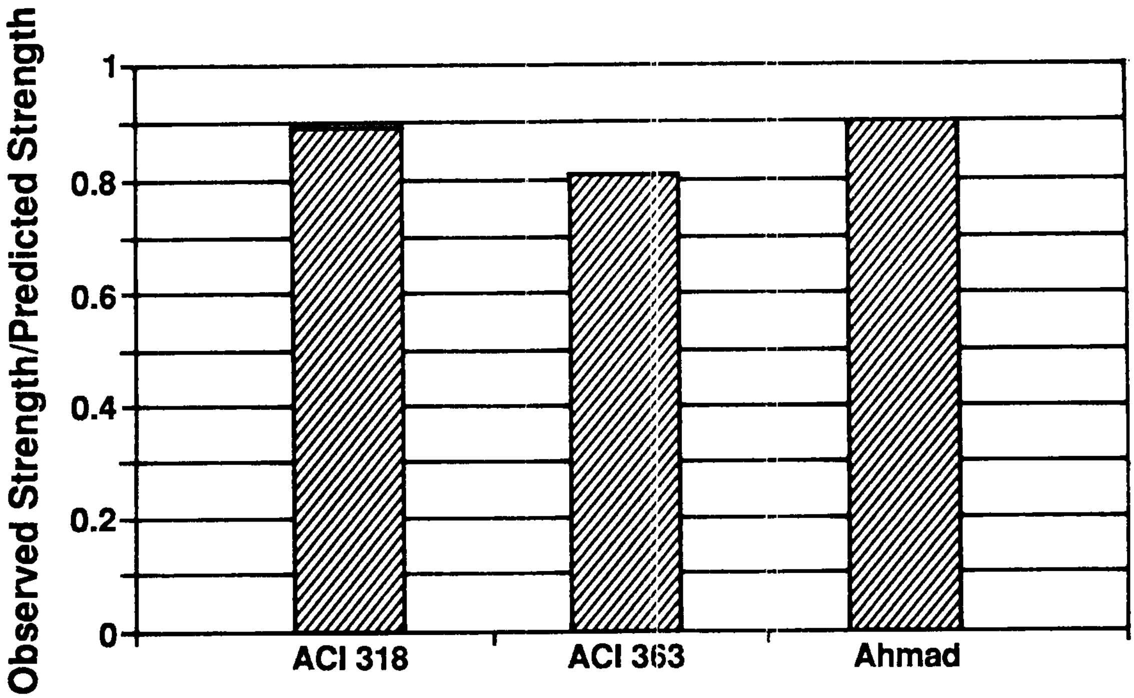

Figure 6.7 Comparison of observed vs. predicted split cylinder strength of HES concrete

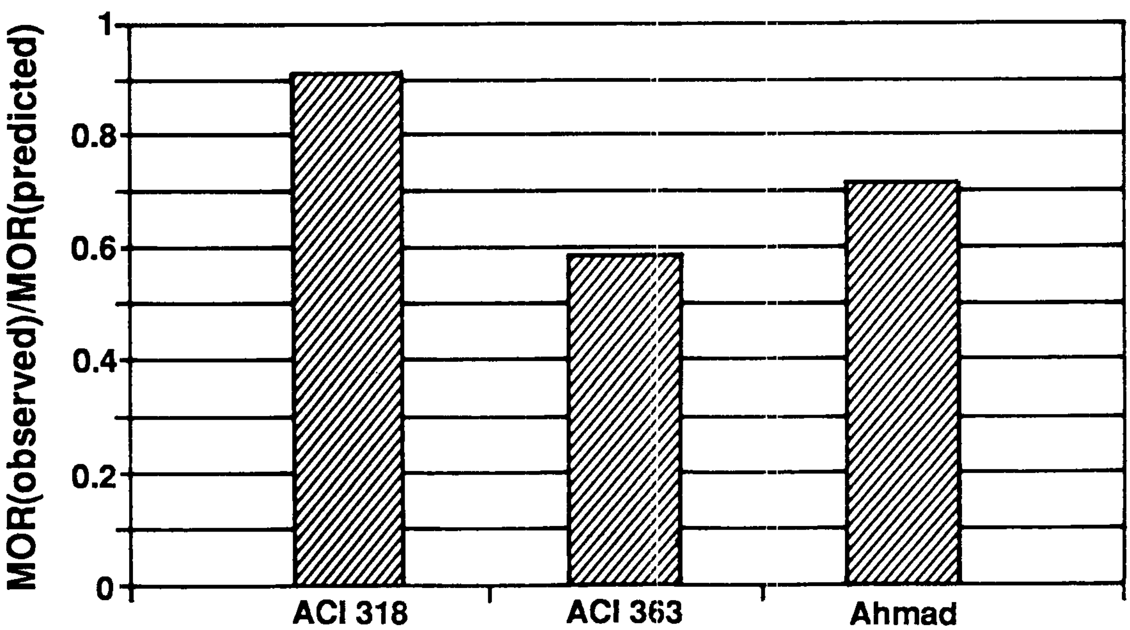

Figure 6.8 Comparison of observed vs. predicted modulus of rupture of HES concrete

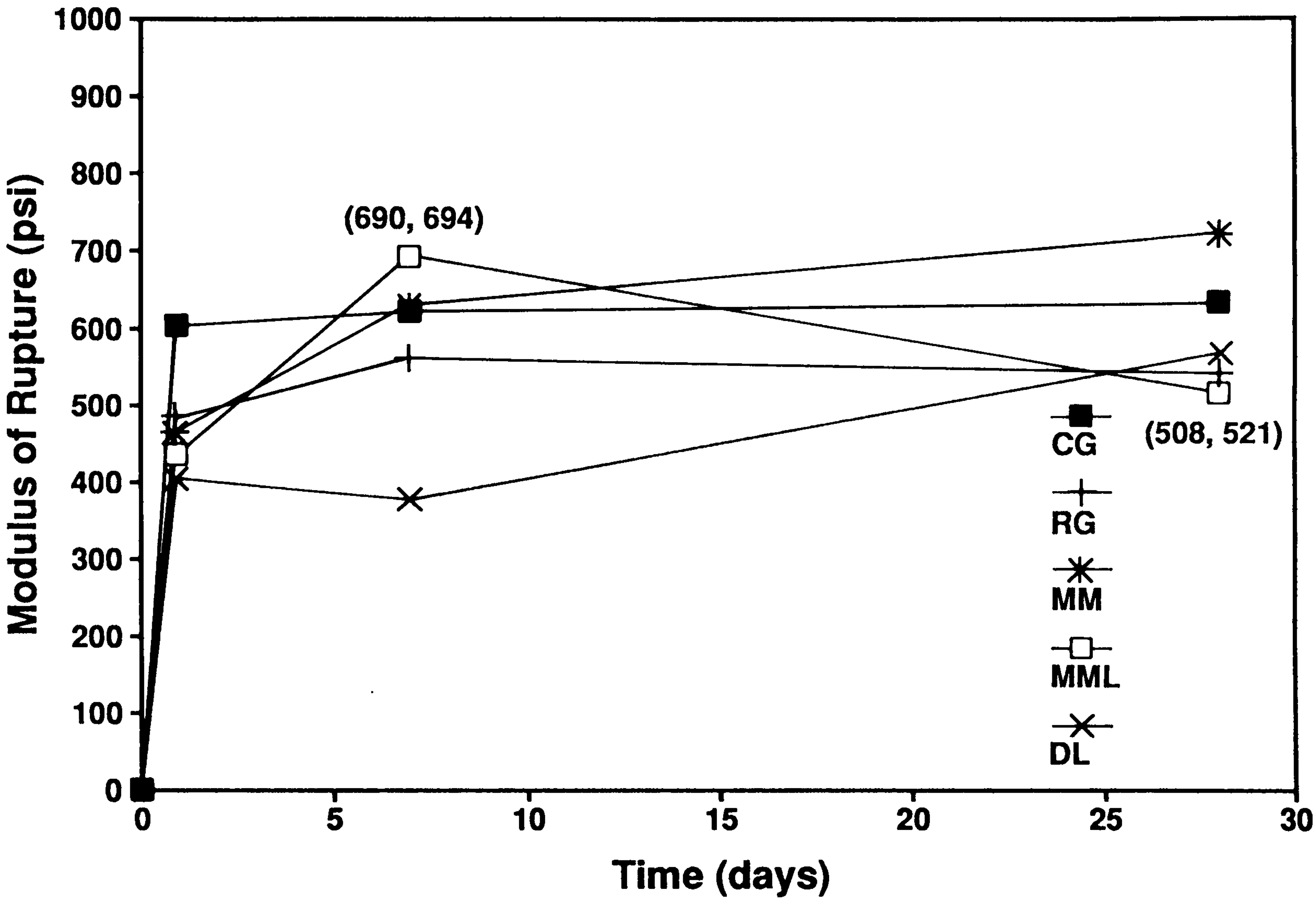

Figure 6.9 Variation of modulus of rupture with time for HES concrete

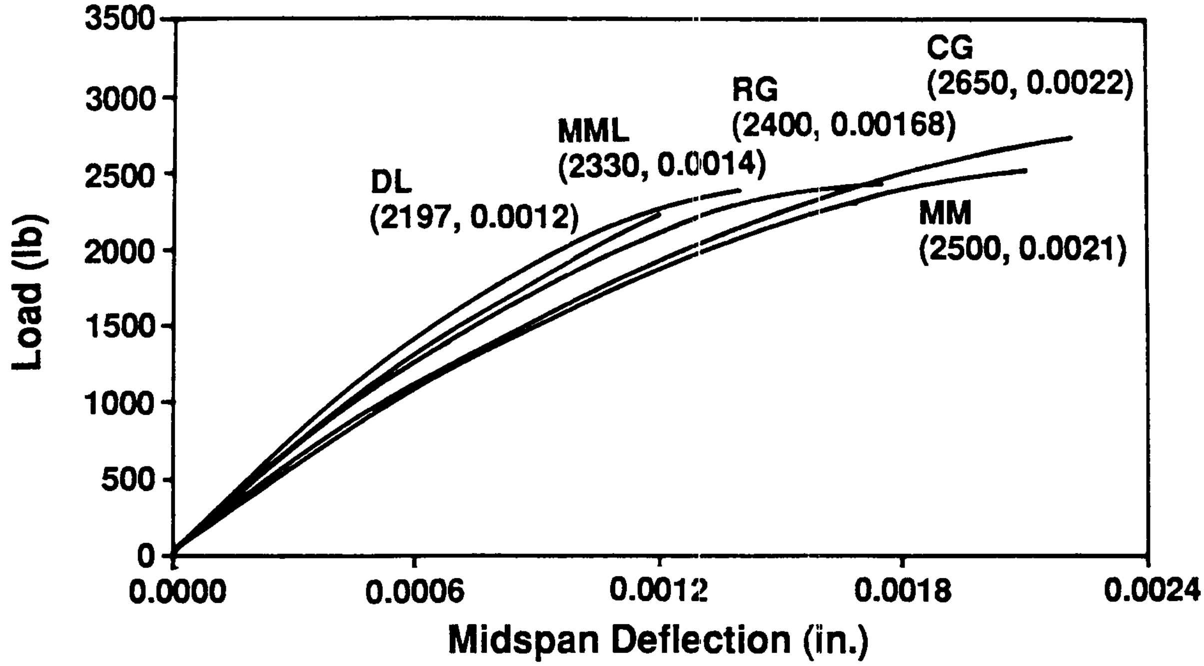

Figure 6.10 Load vs. midspan deflection of HES concrete beam at design age of 1 day

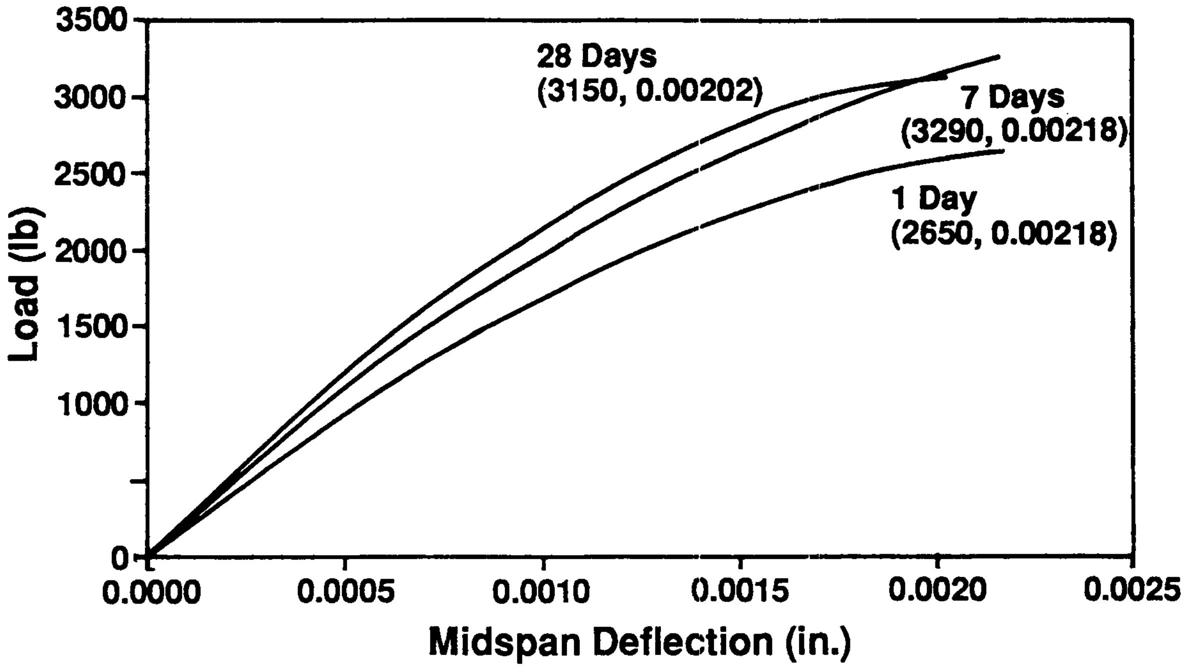

Figure 6.11 Load vs. midspan deflection of HES concrete with CG

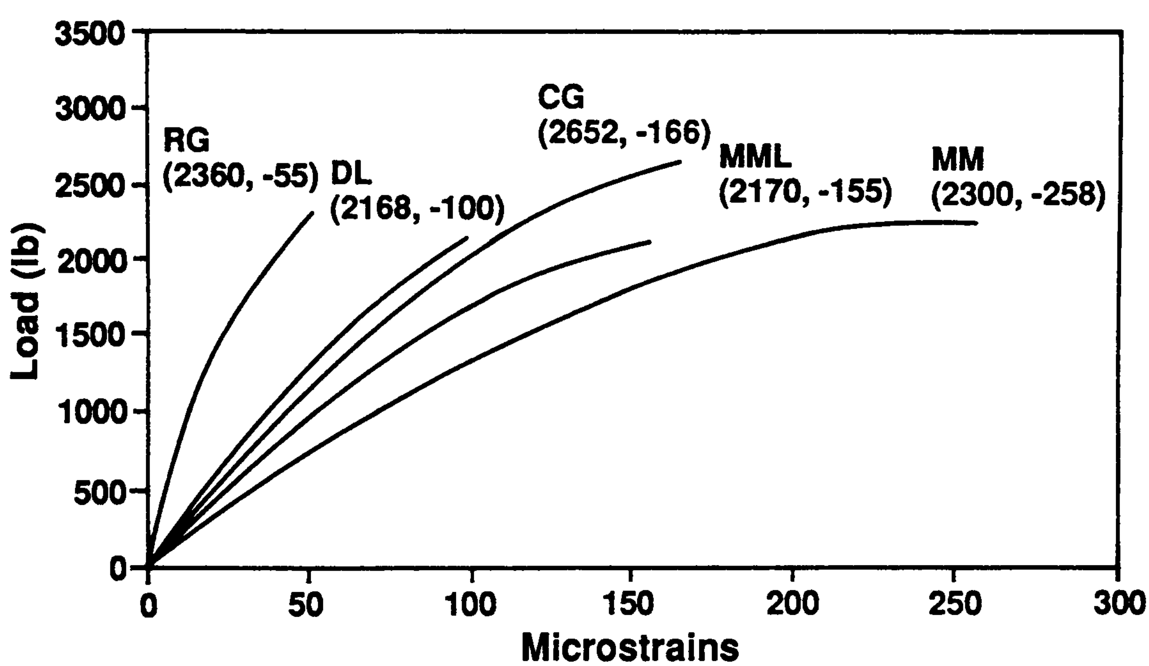

Figure 6.12 Load vs. tensile strain of HES concrete at design age of 1 day

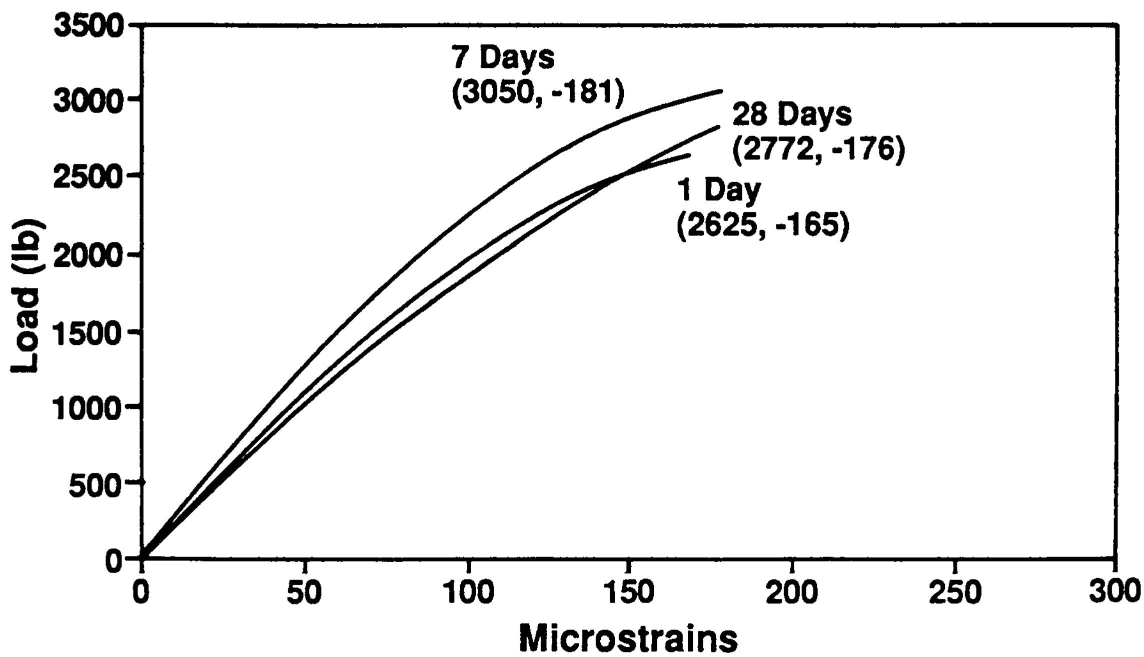

Figure 6.13 Load vs. tensile strain of HES concrete with RG

Two narrow vertical slits were cut in each side of the container to reduce its rigidity. The slits were then filled with silicon to make the container water-tight. Inside the container, small strips of Plexiglas were adhered to the two longitudinal side walls and the bottom to serve as supports for the specimen so that the specimen would be completely surrounded by water. The thickness of the side supports was 1/16 in. (1.6 mm), and that of the bottom supports was 1/8 in. (3.2 mm). These supports ensured proper spacing and clearance of the submerged specimens. The size of the specimen was 3 x 4 x 16 in. (75 x 100 x 400 mm). The specimens were placed inside plastic bags that were filled with water, then placed in the containers. This was to prevent aluminum from reacting with the lime (CaOH) in the concrete.

When the specimens were 14 days old, they were placed, with the finished surface up, in the freezing-thawing chamber and subjected to freezing and thawing between 0° and 40°F (±3°F), or -17.8° to 4.4°C (±1.7°C). Each freezing-thawing cycle required approximately 2.5 hours to complete. The exact amount of time was somewhat influenced by the ambient temperature in the laboratory.

At intervals of not more than 36 freezing-thawing cycles, the specimens were removed from the freezing-thawing chamber, brought to saturated surface dry condition, and weighed in grams. The weight was later automatically converted to pounds by the computer program used for data analysis. The dimensions of each specimen were measured to the nearest 1/8 in. (3.2 mm). Each specimen was then tested for its fundamental transverse frequency.

The fundamental transverse frequency of the specimen was determined using an impact resonance device in accordance with the proposed revision of ASTM C 215-91. The equipment used for the test is shown in Figure A10. The impactor used to excite the specimen for frequency measurement was a 1 x 0.5 x 7.75-in. (25 x 13 x 197-mm) steel bar with a weight of 1.034 lb (469 g). The steel bar was used instead of a spherical impactor for practical convenience. Results obtained using both spherical and prismatic impactors were compared beforehand, and no appreciable difference was found.

Soft rubber pad supports were placed at the estimated nodal points of the specimen, atapproximately 0.224 of the length of the specimen from each end, to allow the specimen to vibrate freely. A small piezometric accelerometer (Model 303A02, PCB Piezotronics, Inc., Depew, New York) weighing 0.113 oz was placed at mid-height on the specimen near one end. The accelerometer had a calibrated operating frequency range of 10 to 10,000 Hz, a self-resonant frequency of 100 kHz, and a voltage sensitivity in excess of 10 mV/g. Amplitude deviation was within 3% full range.

Since the specimen was moist and adhesion to its surface was difficult, the accelerometer was held tightly against the specimen with a rubber band. Soft wax with good acoustic properties was used between the accelerometer and the specimen surface, which frequently was rough, to ensure good mechanical contact. The soft wax also provided some adhesion and kept the accelerometer from slipping.

The specimen was struck lightly at mid-height near its middle in the same direction as the primary axis of the accelerometer. The accelerometer was connected to a power unit supplied by the same manufacturer. The output signal from the power unit was sent to a digital acquisition (DAQ) board in a PC computer. The board (Model AT-MIO-16L-9, National Instruments, Austin, Texas) is a 12-bit, multipurpose, multiple-gain, 16 single-ended or 8 differential channel, input-output, plug-in board with a 9 μs analog-to-digital converter, permitting sampling rates of up to 90,000 to 100,000 samples per second. The board was used within an 80386-based 20 MHz stand-alone microcomputer.

The software used to set up and operate the DAQ board was also provided by National Instruments. This software, LabWindows version 2.0 with the Advanced Analysis Library option, permitted standard programming languages to be used (together with special function calls provided with the software) to control or program data acquisition and to optionally perform analysis on the data and save the data or analysis to computer files. The programming language used in this application was Microsoft QuickBASIC version 4.0.

The program was written so that data acquisition of the output from the accelerometer was self-initiating (i.e., a jump in voltage from the accelerometer would initiate data acquisition). The sampling rate was 50 kHz with 2,048 sample points collected. The program transformed the data to time-based values and, using a LabWindows function call, conducted a fast Fourier transform (FFT) on the first 1,024 data points. The program examined the FFT output and selected the frequency with the largest amplitude as the fundamental or resonant frequency.

The program then used the resonant frequency, in combination with the mass of the specimen, to compute the dynamic modulus of elasticity of the specimen based on the following equation given in ASTM C 215:

$$E = CWn^2$$

where W = weight of specimen, lbs
n = fundamental transverse frequency, Hz
C = 0.00245 (L^3T/bt^3), $\sec^2/\text{in}^2$, for a prism
L = length of specimen, in.
t, b = dimensions of cross section of prism, in (t being in the direction in which the prism is driven)
T = a correction factor obtained from a table given in ASTM C 215 = 1.4 (T depends on the ratio of the radius of gyration to the length of the specimen, and on Poisson's ratio).

The resonant frequency test was conducted three times to observe the repeatability of the results. If any one reading deviated from the average of three measurements by more than 10%, the reading was ignored and the test was repeated. The average of the three final readings was then recorded.

After all the specimens were tested, they were returned to the freezing-thawing chamber and placed at different locations before the freezing-thawing cycles were resumed. Each specimen was subjected to the freezing-thawing test for 300 cycles or until its dynamic modulus became less than 80% of its original value, whichever occurred first.

A set of control specimens kept in water at room temperature was also tested for resonant frequency at the same time as the freezing-thawing specimens. The change in dynamic modulus of the control specimen reflected the effect of strength variation of the concrete.

6.3.2 Specimen Preparation

Five groups of specimens of HES concrete using Type III cement and CG or DL as aggregates were produced for the freezing-thawing tests. Each group consisted of a set of three freezing-thawing specimens and up to two control specimens. In addition to the freezing-thawing specimens, a 6 x 6 x 24-in. (150 x 150 x 600-mm) prism was also cast from which the specimens for rapid chloride permeability and AC impedance tests (Sections 6.5 and 6.6) were later obtained.

The mixture proportions, strength, and plastic properties of the concrete used for the test specimens are detailed in Table 6.8. The concrete was placed in the molds, vibrated with a needle vibrator, and finished with a magnesium float. A thermocouple wire was placed in one of the freezing-thawing specimens to measure the concrete temperature in the freezing-thawing chamber. For curing, all specimens were kept in separate plastic bags at room temperature for 14 days before they were subjected to the freezing-thawing test.

6.3.3 Test Results and Discussions

The results of the freezing-thawing tests expressed in terms of durability factor are summarized in Table 6.9. Durability factor is defined as

$$\text{D.F.} = (E/E_0) \times (N/300)$$

where E_0 = initial dynamic modulus of specimen
N = number of cycles of freezing-thawing up to 300
E = dynamic modulus of specimen after N cycles of freezing-thawing.

Figures 6.14 through 6.17 show the behavior of all test specimens with CG in terms of the effect of freezing-thawing on the relative dynamic modulus of each specimen. The durability factor of all three specimens of C/HE(C)/3 exceeded 100%, indicating that their durability against freezeing-thawing was excellent. The air content of this first group of specimens was 8.9%. Similarly, the durability factor of all three specimens of C/HE(C*)/3 exceeded 80%, indicating that they met the durability criterion established for high performance concrete in this study. The air content of this second group of specimens was 5.4%. On the other hand, all three specimens of C/HE(C)/3F with an air content of 3.9% began to deteriorate quickly after a few cycles of

Table 6.8 Mixture proportions, strength and plastic properties of HES concrete used for freezing-thawing test specimens

		319	307	316	309	486
Reference No.*		**319**	**307**	**316**	**309**	**486**
Batch ID		**C/HE(C)/3**	**C/HE(C)/3F**	**CHALT11**	**C/HE(C*)/3**	**3/HES/1**
Aggregate type		**CG**	**CG**	**CG**	**CG**	**DL**
Sand source		**Lillington**	**Memphis**	**Lillington**	**Lillington**	**Van Buren**
No. of freezing-thawing specimens		**3**	**3**	**3**	**3**	**3**
No. of control specimens		**1**	**2**	**1**	**0**	**1**
Cement (Type III), pcy		870	810	870	870	870
Coarse aggregate, pcy		1,720	1720	1720	1720	1680
Sand, pcy		910	1086	960	1090	1030
HRWR (naphthale-based), oz/cwt		26	18	26	22	15.5
Calcium nitrite (DCI), gal/cy		4.0	6.0	4.0	6.0	4.0
AEA, oz/cwt		1.3	5	9	6	4.2
Water, pcy		300	275	280	275	300
W/C		0.34	0.34	0.32	0.34	0.34
Slump, in		—	1.75	1	2	2.5
Air, %		8.9	3.9	5.3	5.4	3.7
Strength, psi @	1 Day	—	6,390	5,410	5,150	5,920
	14 Days	8,740	9,330	9,510	8,750	7,600
	28 Days	—	10,090	9,670	9,410	7,660
Concrete temperature at placement, ^{O}F		73	75	80	81	85

* Reference No. relates to tables in appendix of volume 2 of this report series.
C/HE(C*)/3 was moist-cured for 14 days before testing.

Table 6.9 Results of freezing-thawing test of HES concrete

Reference No.*	Batch ID	Air Content (%)	No. of Cycles Completed	Durability Factor (%)		
				Specimen 1	Specimen 2	Specimen 3
319	C/HE(C)/3	8.9	300	105	103	105
307	C/HE(C)/3F	3.9	68	15	15	9
316	CHALT11	5.3	300	-	104	40
309	C/HE(C*)/3	5.4	300	82	85	84
486	DL	3.7	310	119	116	117

* Reference No. relates to tables in appendix of volume 2 of this report series.
C/HE(C*)/3 was moist-cured for 14 days before testing.

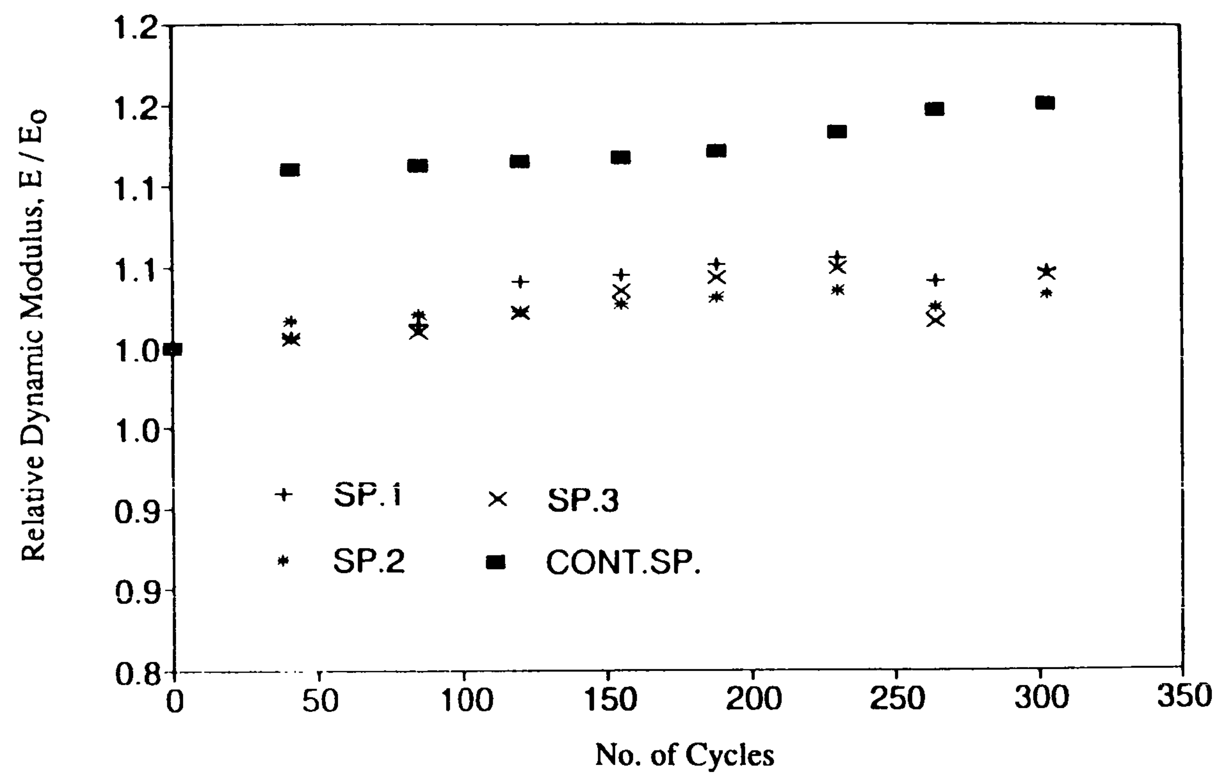

Figure 6.14 **Relative dynamic modulus vs. number of freezing-thawing cycles for C/HE(C)/3**

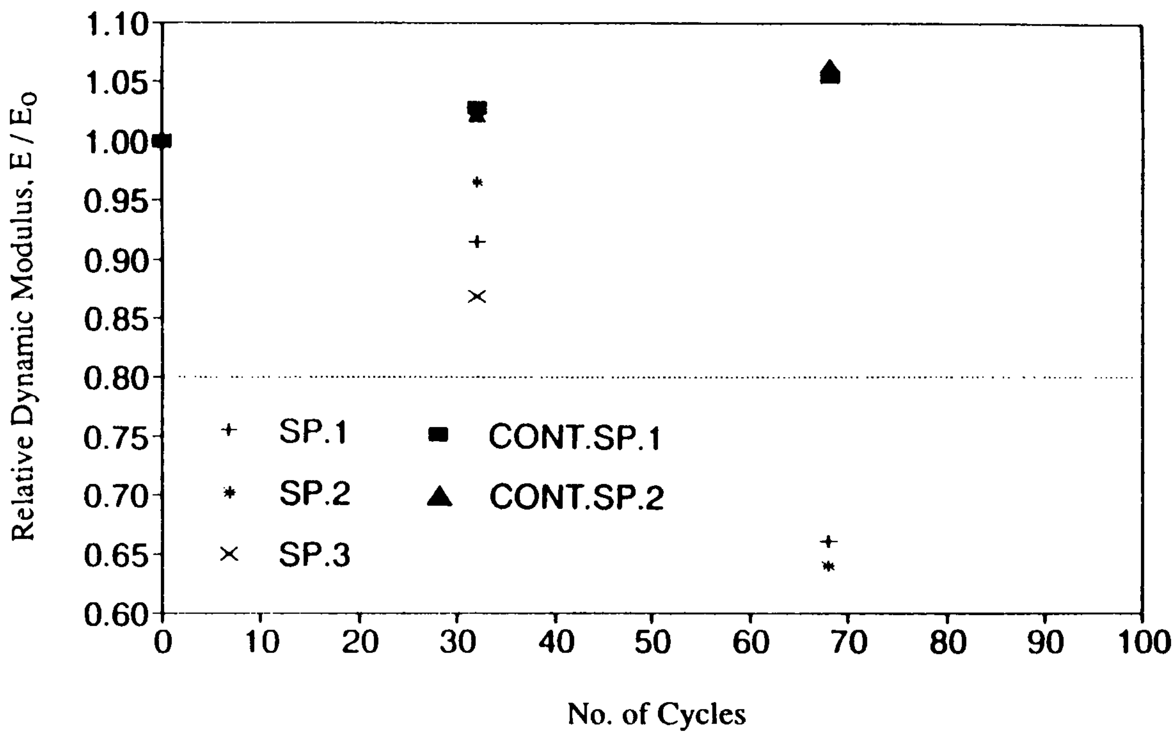

Figure 6.15 **Relative dynamic modulus vs. number of freezing-thawing cycles for C/HE(C)/3F**

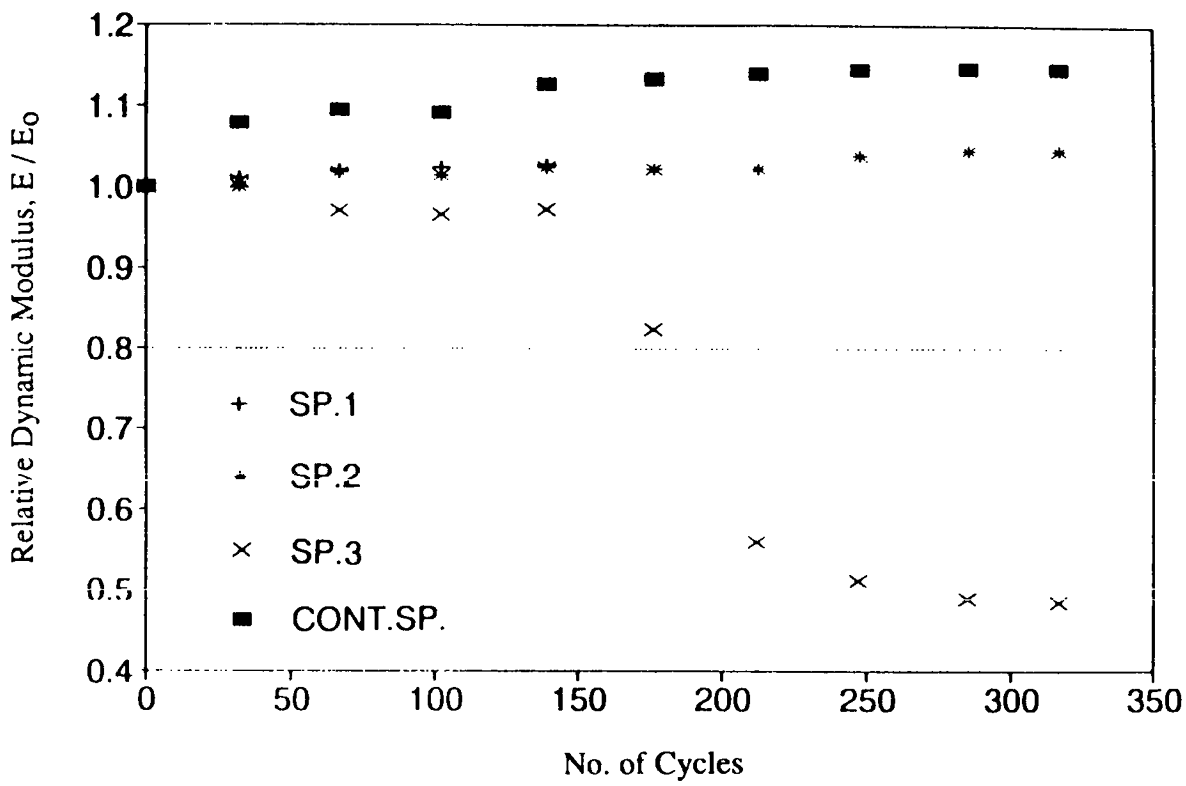

Figure 6.16 Relative dynamic modulus vs. number of freezing-thawing cycles for CHALT11

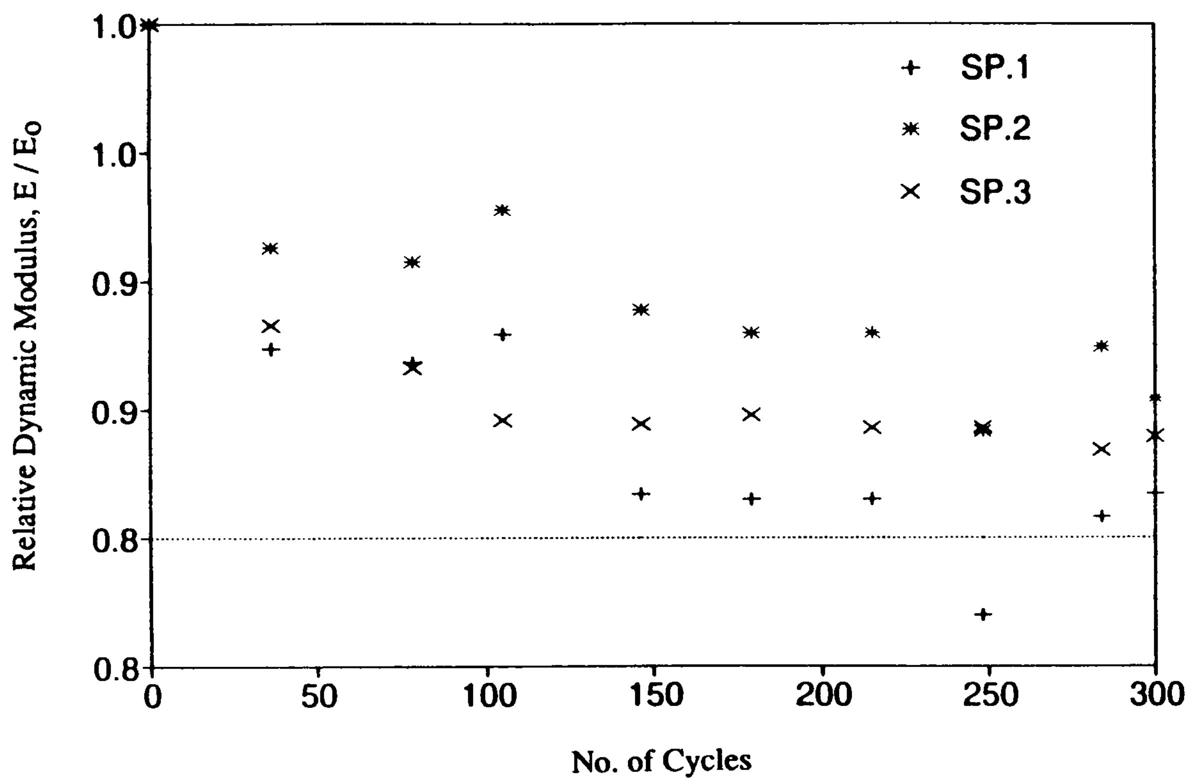

Figure 6.17 **Relative dynamic modulus vs. number of freezing-thawing cycles for C/HE(C*)/3**

freezing-thawing, and their durability factors were reduced substantially below 80% after only 68 cycles of freezing-thawing.

Comparison of the air-cured C/HE(C)/3 series with the moist-cured C/HE(C*)/3 series, indicates that moist curing might reduce the durability factors of HES concretes tested by the rather severe procedure of ASTM C 666. Moist curing prior to freezing-thawing testing may have a dual effect. First, moist curing provides additional water to react with unhydrated cement, thus yielding more hydrated cement paste, smaller voids, and stronger and less permeable concrete. Second, moist curing does not allow the concrete to dry out before freezing, thus causing more frost damage.

For the CHALT11 series, Specimen 2 showed excellent durability with a durability factor of 104% after 300 cycles of freezing-thawing. Specimen 1 behaved just like Specimen 2 for the first 150 cycles of freezing-thawing. Unfortunately, the specimen was accidentally broken and the test had to be discontinued. Specimen 3 experienced some degradation for the first 150 cycles of freezing-thawing, as evidenced by the small reduction in its relative dynamic modulus. After 200 cycles of freezing-thawing, significant deterioration seemed to have occurred in the specimen resulting in a reduction of its durability factor to 49%, but the behavior of the specimen became stabilized for additional freezing and thawing up to more than 300 cycles. The air content of the concrete was 5.3%.

The three specimens of 3/HES/1 with DL, tested at Arkansas, produced the best performance in freezing-thawing resistance even though their air content was only 3.7%

From the results of these tests, it still seems appropriate to suggest that HES concrete should have a minimum air content of 5% in order to ensure its enhanced frost durability, despite the superior performance of the Arkansas tests.

6.4 Shrinkage Tests

The shrinkage tests were conducted on test specimens of 4 x 4 x 11.25-in. (100 x 100 x 281-mm) prisms for a period of up to 90 days. Three replicate specimens were tested for each type of coarse aggregate. The overall program for the shrinkage tests is outlined in Table 6.3.

6.4.1 Test Setup and Procedure

The shrinkage tests were conducted in accordance with ASTM C 157. A digital dial gage (OKO SOKKI model DG 154) was used to measure length changes. The gage had a range of 2 in. (50 mm). A view of the shrinkage test setup is shown in Figure A.11.

6.4.2 *Specimen Preparation*

The 4 x 4 x 11.25-in. (100 x 100 x 281-mm) prisms were cast in steel molds, internally vibrated with a needle vibrator, and finished with a magnesium float. The specimens with molds were protected from moisture loss by covering them with plastic sheets. The specimens were stripped at the design age of 24 hours. Upon removal of the specimens from the molds, the specimens and the standard calibration bar were placed in water maintained at 73°F for a period of 30 minutes before being measured for length. This was done to equalize the temperature of the specimens and the bar.

After the initial readings, the specimens were cured in lime-saturated water for 28 days. During this period, the specimens were removed from the water only for measurements. Subsequently, they were stored in air under normal laboratory condictions.

6.4.3 *Test Results and Discussions*

The results of the shrinkage tests are summarized in Table 6.10. They indicate that at 90 days, more shrinkage strains occurred for the HES concrete with CG than for HES concretes with the other types of coarse aggregate.

The variation of the shrinkage strains with time for different types of coarse aggregates is shown in Figure 6.18. These curves are average of three replicate specimens, except for HES concrete with RG (due to experimental error). It can be seen that three of the four curves show an initial expansion where readings were taken at early ages. This phenomenon is common for shrinkage specimens stored in water as described above. For HES concrete with CG shrinkage strains were rather small for the first 40 days but increased gradually to about 300 microstrains at 90 days. The general trend of variation of shrinkage strains with time for HES concrete was similar to that generally accepted for conventional concrete.

Note that the coarse aggregate in a concrete provides a restraining effect on the drying shrinkage of pure cement paste. The amount of restraint provided by the aggregate depends on the amount of coarse aggregate, the stiffness of the concrete and the maximum size of the coarse aggregate (Mindess and Young 1981). The stresses at the cement-paste aggregate interface due to drying shrinkage increase as the maximum aggregate size increases. The size and shape of a concrete specimen determine the rate of moisture loss and hence the rate and the magnitude of drying shrinkage. The absorption characteristics of the coarse aggregate also influence the shrinkage characteristics of the concrete, especially during early ages. The length of diffusion path also has a strong influence on the rate of moisture loss, which in turn affects the shrinkage characteristics.

ACI Committee 209 (1993a) recommends a set of empirical equations that allow shrinkage strains to be estimated as a function of drying and relative humidity. The value of the ultimate shrinkage strain recommended by ACI Committee 209 is 730 x 10^{-6} in./in. The magnitude of ultimate shrinkage strain is difficult to estimate accurately because it depends on a number of factors including W/C, degree of hydration, presence of admixtures, and cement content. The

Table 6.10 Summary of shrinkage test results for HES concrete

Coarse Aggregate Type	Batch ID	Age (days)	Shrinkage Strain (microstrains)	Avg. of No. of Specimens
MM	M/HE(C)/3	1	0	3
		14	-33	
		30	-2	
		80	308	
		90	351	
		112	446	
CG	C/HE(C)/3	1	0	3
		42	138	
		89	479	
		90	481	
		144	564	
RG	R/HE(C)/3	1	0	1
		14	-22	
		28	-39	
		77	177	
		90	201	
		109	236	
CG (std)	C/HE(C*)/3	1	0	3
		15	-27	
		44	-13	
		60	282	
		90	308	
		230	427	
		285	466	
DL	3/HES/2	1	0	2
		8	.5	
		15	65	
		28	500	
		60	6.5	
		90	690	

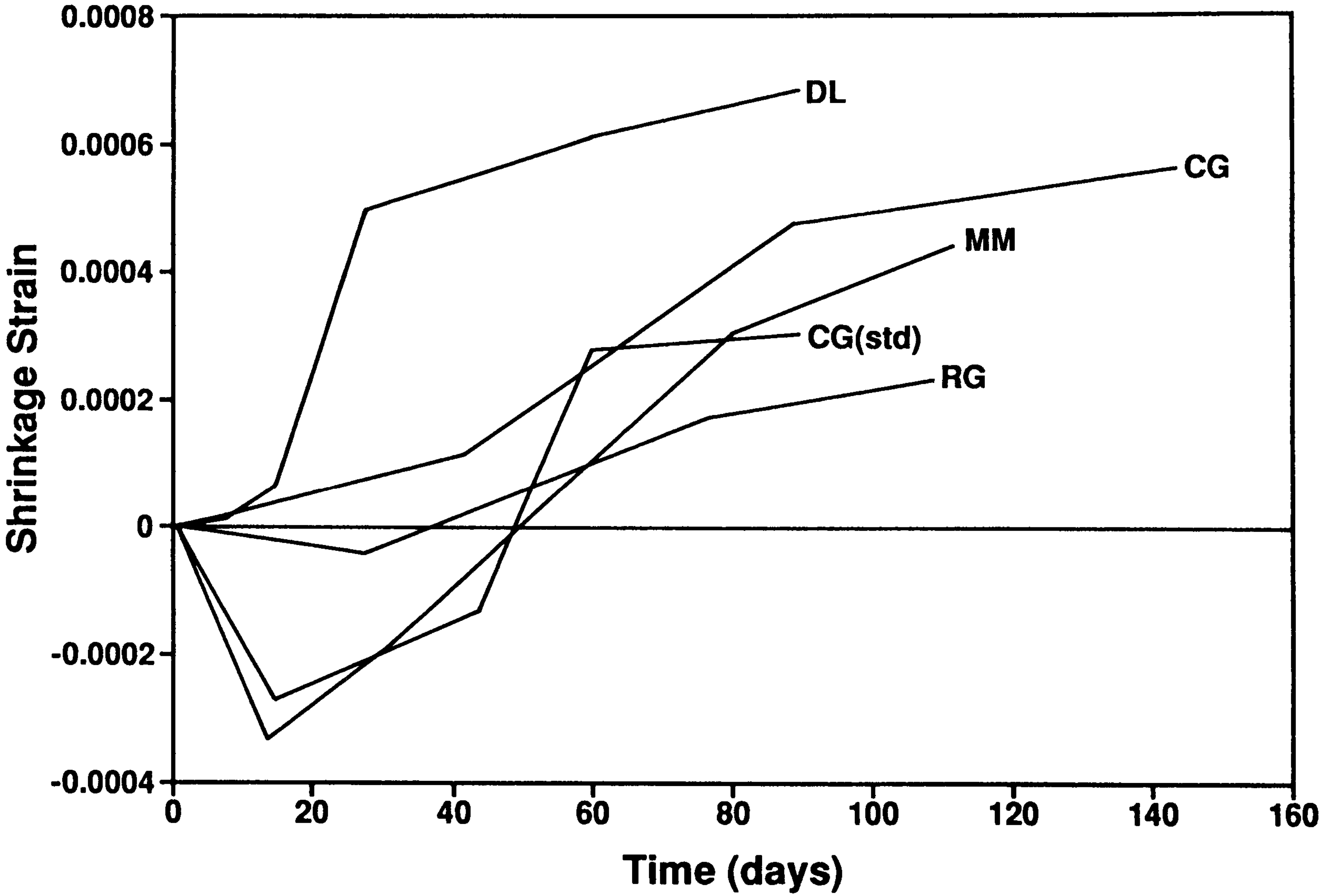

Figure 6.18 Variation of shrinkage strain with time for HES concrete

results of this study indicate that average shrinkage strains at 90 days varied from 210 microstrains (for HES concrete with RG) to 481 microstrains (for HES concrete with CG).

6.5 Rapid Chloride Permeability Tests

6.5.1 Test Setup and Procedure

The rapid chloride permeability test (RCPT) was performed in accordance with AASHTO T 277-83 and the identical ASTM C 1202. The typical specimen was a 2 in. (50 mm) thick slice sawed from the top of a 3.75 in. (94 mm) diameter core from a concrete prism cast at the same time as the freezing-thawing test specimens (Section 6.3.2). The top of the specimen was the finished surface that would be exposed to chloride ions.

The specimen was prepared by vacuum saturation and soaking in water for 18 hours, as shown in Figue A.12. The specimen was then sealed in the test cells. The cell connected to the top surface of the specimen was filled with 3% sodium chloride (NaCl) solution, and the cell connected to the bottom surface of the specimen was filled with 0.3 N sodium hydroxide (NaOH) (Figure A.13). Two replicate specimens were always tested concurrently. Both specimens were energized continuously with 60v direct current between the cells (Figure A.14). The current flowing through the concrete specimen was measured and plotted on a strip chart recorder for 6 hours (Figure A.15). A switching unit was built to allow two measurements to be alternately plotted on one strip chart. The total time required to complete each test was about 33 hours. The test procedure followed these steps:

1. Obtain two 3.75 in. (94mm) diameter concrete samples by coring through a 6 x 6-in. (150 x 150-mm) prism, oriented as cast, when the concrete is 12 days old. (Two days are allowed for specimen preparation so that actual testing can be started at 14 days.) Keep the samples in a plastic bag.

2. Saw a 2-in. (50-mm) slice from the top of each core. Surface-dry the specimens for 10 minutes after cutting. Mark ID on each specimen and clearly identify its top surface. Keep the specimens in a plastic bag.

3. Boil 2 liters of water vigorously to de-air the water. Do not cap the boiling pot. Allow the water to cool.

4. Prepare clear epoxy; brush on side surface of each specimen. Allow the epoxy to cure per manufacturer's instructions (use a type that will cure in about 1 hour).

5. Place both specimens, separated slightly, in a tilted position in a beaker, then place beaker in a *vacuum-type* desiccator and start vacuum (1 mm Hg abs.). Maintain vacuum for 3 hours.

6. While keeping the vacuum pump running, fill the beaker with the boiled water until it submerges the specimens.

7. Continue running the vacuum pump for 1 additional hour.

8. Turn the pump off and allow air to enter the desiccator.

9. Allow both specimens to remain in the beaker to soak in water for 18 hours.

10. Remove the specimens from the beaker, blot off water, and place the specimens in a plastic bag.

11. Place a small amount of silicon sealant on the brass shim around the inside perimeter of each cell. Place the specimen in the cell assembly with its top side toward the NaCl cell. Apply liberal amounts of silicon sealant around the specimen to seal it to the cells. Allow the sealant to cure per manufacturer's recommendations (use a type that will cure in about 1 hour).

12. Fill (–) cell (top side) with 3% NaCl solution. Fill (+) cell with 0.3 N NaOH. Connect wires with negative to NaCl and positive to NaOH. Turn on power, set to 60.0±0.1v direct current, and turn on plotter. Set plotter at 1v full scale (this is actually 0.985 A full scale on NCSU's plotter) and 2 in./hr (50 mm/hr). Record time and ID number on plotter paper. Run the test for 6 hours.

13. Remove the specimens from the test cells. Rinse the cells and clean the assembly.

14. Integrate the area under the curve produced by the plotter in units of ampere-seconds (i.e., coulombs) and record results.

6.5.2 Test Results and Discussions

A total of eight groups of specimens of HES concrete were subjected to the RCPT (Table 6.11). Each group consisted of two replicate specimens that were tested concurrently in two separate RCPT cells. A typical output strip chart from the RCPT is shown in Figure A.16. The abscissa represents time, obtained at a chart rate of 5 cm/hr for approximately 6 hours. The ordinate represents current (in amperes) flowing through the specimen, with a chart calibration of full scale being equal to 0.985 A for the particular plotter used.

As described above, two replicate specimens were tested together and both were energized continuously for 6 hours. However, the current flowing through each specimen was measured and plotted alternately on the strip chart. The average area under the curve in ampere-seconds (coulombs) is the total charge that passed through the specimen in 6 hours. The initial current in amperes is the average ordinate just after the test was started.

Table 6.11 Results of rapid chloride permeability test (RCPT) of HES concrete

				Freeaing-	Durability Factor (%)			RCPT	
Ref. No.*	Batch ID	W/C	Air %	Thawing Cycles	Sp. 1	Sp. 2	Sp. 3	Initial Current (A)	Total Charge (C)
319	C/HE(C)/3	0.34	8.9	300	105	103	105	0.131	3,670
307	C/HE(C)/3F	0.34	3.9	68	15	15	9	0.146	4,390
316	CHALT11	0.32	5.3	300	-	104	40	0.163	4,980
309	C/HE(C*)/3	0.34	5.4	300	82	85	84	0.160	4,904
359	M/HE(C)/3	0.32	6.7	-	-	-	-	0.212	6,840
360	M/HE(L)/3	0.34	2.3	-	-	-	-	0.187	5,880
408	R/HE(C)/3	0.34	2.0	-	-	-	-	0.110	3,260
486	3/HES/1	0.34	3.7	310	119	116	117	0.275**	1,687**

* Reference No. relates to tables in appendix of volume 2 of this report series.
** Average of two samples.
C/HE(C*)/3 was moist-cured for 14 days before testing.

It can be seen that the amount of current flowing through the specimen tended to increase during the RCPT, due at least in part to the temperature increase in the specimen. This heating problem was, in turn, caused by the current. Thus the question was raised whether the initial current, which is also an indirect measure of the concrete conductance, could be used in lieu of the total charge as a more convenient way to obtain data for the RCPT. If so, 6 hours of testing time could be saved. It is encouraging to observe that a comparison of the initial current with the total charge for each of the eight groups of testing (Table 6.11) indicates a good correlation between the two measurements.

Note that both the freezing-thawing test and the RCPT were performed for five of the eight groups of specimens summarized in Table 6.11. As discussed in Section 6.3.3, four of the five groups — C/HE(C)/3, CHALT11, C/HE(C*)/3, and 3/HES/1 — showed excellent frost durability. However, the results of the RCPT would place the five groups in the "low" to "high" permeability categories according to the classifications of ASTM C 1202. The RCPT essentially measures ionic permeability; when the test is applied to high performance concrete in which many additional ions are introduced from the various admixtures, the added ions may cause the concrete to be more conductive electrically and make it *appear* to be more permeable than it really is. The higher coulomb value does not mean lower resistance to chloride ion penetration (Berke et al. 1988, ASTM 1992).

6.6 AC Impedance Tests

The objective of conducting the AC impedance test was to examine its potential as a reasonable alternative to the RCPT for more rapid determination of concrete permeability. The goal was to explore a faster method that could be more versatile and portable. The method would also avoid elaborate test setups and specimen preparation as well as expensive and time-consuming procedures. The use of external measurement terminals would allow more versatility than the internal terminals used for the RCPT. A "point source" type of test such as the AC impedance test would also be more versatile since it could be adapted more easily to specimens of different shapes.

The RCPT measures total conductance of a specimen rather than conductivity. Similarly, the AC impedance test measures total resistance (in ohms) which may be more indicative of the gross concrete properties than resistivity. As long as the test could classify concrete as accurately as the RCPT, it would be a satisfactory alternative.

6.6.1 Test Setup and Procedure

Currently there are no AASHTO or ASTM standards for the AC impedance test. The test for this investigation was conducted using a Kohlrausch bridge instrument (Model 4000, AEMC Instruments, Boston). The Kohlrausch bridge uses 1,000 Hz AC with an accuracy of 1% on the impedance measurement. Eight impedance ranges can be selected using pushbuttons; usually the 1-10 kΩ range was used. The voltage actually placed across the specimen was measured to be about 0.20v. Six 1.5v AA batteries were used as the power supply. The general setup is shown in Figure A.17.

A good electrical connection between the Kohlrausch bridge terminals and the concrete specimen was ensured by using potassium agar gel. The impedance (resistance) was measured at five random points on each specimen (see Figure A.18).

The AC impedance test was conducted in conjunction with the RCPT at three different stages:

(1) after the RCPT specimens were cut, but before they were subjected to the vacuum saturation process;
(2) after the specimens were vacuum-saturated, but before they were subjected to RCPT;
(3) after the specimens were subjected to RCPT.

Therefore the specimens for the AC impedance test were prepared in exactly the same manner as for the RCPT. The AC impedance test procedure followed these steps:

1. After the specimen is cut from a 3.75-in. (94-mm) concrete core, air-dry the specimen to avoid wet surfaces that may distort the electrical measurements.

2. Connect two electrode wires to the Kohlrausch bridge (which is similar to a Wheatstone bridge), with the specimen placed into one arm of the bridge. The free end of each wire should have a flattened piece of solder (about 1/4 in. [6 mm] diameter) as electrode. Virtually any type of electrical wire can be used to connect the concrete specimen into the bridge circuit since the current is very small. A wire length of about 18 in. (450 mm) is recommended.

3. Put a rubber glove on the hand that will press the electrodes onto the concrete specimen.

4. Dip the solder ends of the wires into potassium agar gel to obtain a small amount of gel, which ensures good conductivity between the electrodes and the concrete.

5. Press the solder flat ends against the two flat surfaces of the test specimen and hold them in place with the thumb and middle finger of one hand. With a little practice, a good connection can be made by feel. Also, the gel "fingerprint" should be kept small and consistent throughout the testing.

6. Balance the Kohlrausch bridge circuit and record the impedance measurement (in ohms).

7. Repeat the measurement at five random locations on the flat surfaces of each of the replicate specimens. Be careful to not allow the gel fingerprints to overlap, as this will affect the conductance.

6.6.2 Test Results and Discussions

AC impedance tests were conducted along with the RCPT on four groups of HES concrete specimens (Table 6.12). (The AC impedance test was not conducted at Arkansas on concrete with DL.) As mentioned above, AC impedance was measured five times on each of two replicate specimens, for a total of 10 measurements. Therefore each impedance value listed in Table 6.12 is the average of the 10 measurements.

The AC impedance was measured three times on the RCPT specimens: (1) just after the specimen was cut but before vacuum saturation, (2) after vacuum saturation of the specimen, and (3) after the RCPT was completed. Note that the degree of saturation is initially more varied in a specimen, so its conductance is lower and its impedance higher. After vacuum saturation, the degree of saturation of the specimen becomes more uniform, so its conductance is improved and its impedance reduced. After the RCPT is conducted on the specimen, its impedance again becomes slightly higher; this is because the test has affected the conductance of the specimen by driving some unknown amount of chloride ions into the concrete and driving out some of the ions originally in the pore water of the concrete. Furthermore, any heating of the specimen may accelerate further hydration of residual cement, thereby changing the electrical conductance pathways through the concrete. Because of these effects, vacuum saturation is regarded as the best preparation of the specimen for the AC impedance test.

Table 6.12 Results of AC impedance test of HES concrete

				AC Impedance Test (Ω)			RCPT	
Reference. No.[1]	Batch ID	W/C	Air (%)	Before Sat.[2]	After Sat.[3]	After Test[4]	Initial Current (A)	Total Charge (C)
319	C/HE(C)/3	0.34	8.9	3,055	1,627	1,762	0.131	3,670
359	M/HE(C)/3	0.32	6.7	2,388	1,280	1,290	0.212	6,840
360	M/HE(L)/3	0.34	2.3	1,814	1,550	1,641	0.187	5,880
408	R/HE(C)/3	0.34	2.0	2,318	1,890	2,130	0.110	3,260

[1] Reference No. relates to tables in Aapendix of volume 2 of this report series.
[2] Test conducted before specimen was vacuum-saturated for RCPT.
[3] Test conducted after specimen was vacuum-saturated for RCPT.
[4] Test conducted after completion of RCPT.

As discussed before, chloride permeability (in coulombs) is essentially a conductance test whereas AC impedance (in ohms) is essentially a resistance test. Since the two properties are reciprocals, an inverse relationship could be expected between the data from these measurements. After evaluating the two sets of data in several ways, it was determined that the best way to correlate the two properties is by expressing the inverse impedance (reciprocal of impedance) in terms of the initial current as shown in Figure 6.19.

6.7 Concrete-to-Concrete Bond Tests

Concrete-to-concrete (C-C) bond tests were conducted to determine the interfacial bond strength and the corresponding interfacial deformation. For these tests, the specimen was designed so as to have a direct shear condition at the interface between the two segments of the specimen. Each specimen used in the tests was composed of an inverted L-shaped Segment A bonded to an upright L-shaped Segment B (see Figure A.19). A standard North Carolina Department of

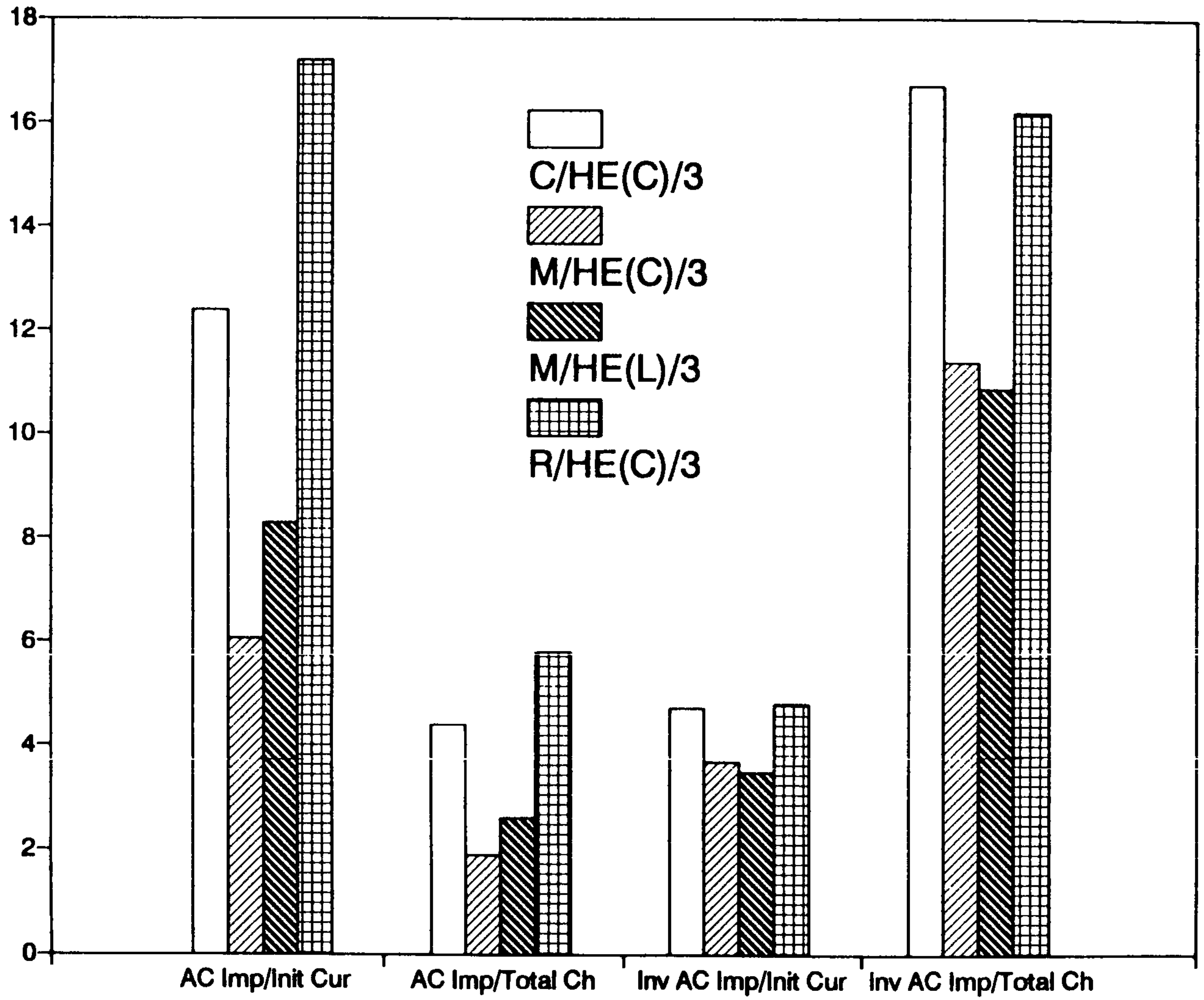

Figure 6.19 Comparison of results from AC impedance test and RCPT

Transportation (NCDOT) concrete mixture termed the "control" mixture was used for casting Segment A, and HES concrete was used to cast Segment B. This method of fabricating the test specimen simulated the condition of a high performance concrete overlay on an existing pavement of "control" concrete.

6.7.1 Test Setup and Procedure

The direct shear tests were conducted in an 810 Material Test System (MTS) with a 200 kip (890 kN) capacity. The test was conducted at the design age (1 day) of the HES concrete. The age of the "control" concrete segment of the C-C bond specimen was well over 100 days.

The load was applied at the rate of 1,500 lbs/min (6.68 kN/min). During the test, the interfacial deformation along the shearing plane was measured by two LVDTs, one mounted at the front and the other at the back of the specimen. The LVDTs had a gage length of 4 in. (100 mm) and were attached to angles that were then glued to the specimen. The load output and the outputs from the two LVDTs were recorded at 2-second intervals using the OPTIM data acquisition system (Megadec 100).

For safety reasons, wood blocks slightly less than 2 in. (50 mm) thick were inserted into the gaps between Segment A and Segment B of the test specimen. A chain was also used to loosely tie the lifting handles in the top segment of the test specimen to the loading head of the test machine. This was done to ensure that the top segment of the test specimen would not fall after failure. A pictorial view of the test setup is shown in Figure A.20.

6.7.2 Specimen Preparation

The direct shear L-shaped segments were reinforced with mild steel bars and were provided with lifting handles. The reinforcement in each L-shaped segment of the specimen consisted of two #2 bars and two #3 bars (see Figure A.19). The segment that used the "control" concrete (i.e., Segment A) was cured in a moist room for 28 days. After Segment A was air-dried, its bonding surface was sandblasted using extra-fine sand under 90 psi (0.62 MPa) pressure for a period of 20 seconds to simulate field conditions for an overlay installation. Segment A was then placed on its side with the bonding surface facing upward. Prior to casting Segment B, a wet cloth was placed on the sandblasted surface for 30 minutes. Segment B was cast using the HES concrete.

After casting, the C-C bond specimens were covered with plastic sheets to protect them from moisture loss. The test specimens were cured until the time of testing. Companion 4 x 8-in. (100 x 200-mm) cylinders were also cast when casting Segments A and B of the test specimens. The curing conditions of these companion cylinders were identical to those of Segments A and B.

6.7.3 Test Results and Discussions

The C-C bond strength test results are summarized in Table 6.13. The strength values of the NCDOT "control" concrete and HES concrete reported in the table are the average compressive

Table 6.13 Summary of test results for bond tests of HES concrete

Type of Test	Coarse Aggregate Type	Specimen ID	Age at Testing	Air Content (%)	Slump (in.)	Concrete Temp. (^{o}F)	f_c' of HPC at Testing (psi)	f_c' of NCDOT "control" Mix at 28 Days (psi)	f_c of NCDOT Mix at Testing (psi)
	MM	HE(MM)/5A	24 Hours	2.4	6	74	5,420	6180	7,100 (123 Days)
	MM	HE(MM)/5B	24.5 Hourrs	2.6	6	74	5,440	6180	7,300 (123 Days)
C-C	CG	HE(CG)/5A	24 Hours	2.8	1.4	73	6,320	6170	7,150 (117 Days)
Bond Test	CG	HE(CG)/5B	26 Hours	3.0	1.6	73	6,340	6190	7,250 (117 Days)
	CG	NCDOT(CG)	7 Days	5.8	3.4	69	4,170	6175	7,300 (137 Days)
	CG		7 Days	6.0	3.6	69	4,190	6185	7,500 (137 Days)
C-S	CG	HE(CG)/5A	24 Hours	5.9	4.5	80	5,650	NA	NA
Bond Test	CG	HE(CG)/5B	24 Hours	6.5	6	79	6,100	NA	NA

strength of two 4 x 8 in. (100 x 200 mm) replicate specimens. Graphs of load vs. interfacial deformation for HES concrete with CG and MM are shown in Figure 6.20. The result for each test specimen is shown separately. For specimens with a relatively longer test age, the load-interfacial deformation response curves indicate that stiffness increased and remained linear up to a larger strain value. Although these results are very limited, they seem to show that interfacial bond strength is to be very sensitive to age, especially during early ages.

The nominal bond stress between the control concrete and HES concrete can be computed by dividing the maximum debonding load by the nominal bonding area of 36 in^2 (232 cm^2). The values of the nominal C-C bond stress range from 275 psi (1.9 MPa) for HES concrete with CG to 350 psi (2.4 MPa) for HES concrete with MM.

A control specimen, with both the L-shaped segments A and B made from standard NCDOT mix, was also fabricated and tested at 7 days. Figure 6.20 also shows the results for the control specimen. Note that the curved portion at the top of the load-interfacial deformation response curve is due to interfacial slip, which occurred just prior to the failure of the specimen. The nominal C-C bond stress in this case is 330 psi (2.3 MPa).

6.8 Concrete-to-Steel Bond Tests

6.8.1 Test Set-Up and Procedure

The concrete-to-reinforcing steel (C-S) bond test specimens had a cross-sectional area of 6 x 15 in.(150 x 375 mm) with a length of 30.2 in. (755 mm) for HES concrete. The test frame and general loading arrangement for the C-S bond test is shown in Figure A.21. A 120 kip (534 kN) center hole jack was used to apply the load to a #6 reinforcing bar embedded in the concrete specimen. The load was increased monotonically and the associated slips at the loaded and free ends of the bar were monitored. The reinforcing bar was axially attached to a 160 ksi (1,102 MPa) smooth prestressing rod of equal diameter. This smooth prestressing rod was instrumented with electrical resistance strain gages to act as a load cell.

Bar slip was measured using dial gages accurate to 0.0001 in. (0.0025 mm). At the free end, the gage was mounted with the probe touching the end of the bar. At the loaded end, the gages were attached to the bar using a ring and set screws.

Loads were applied to the bar in increments of 2 kips (4.5 ksi, 31 MPa stress) until yielding occurred. Loading was stopped either upon pullout of the reinforcing bar or upon reaching 125% to 140% of the yield strength of the bar. At each load increment, load was maintained for a short period until the slip movements stabilized.

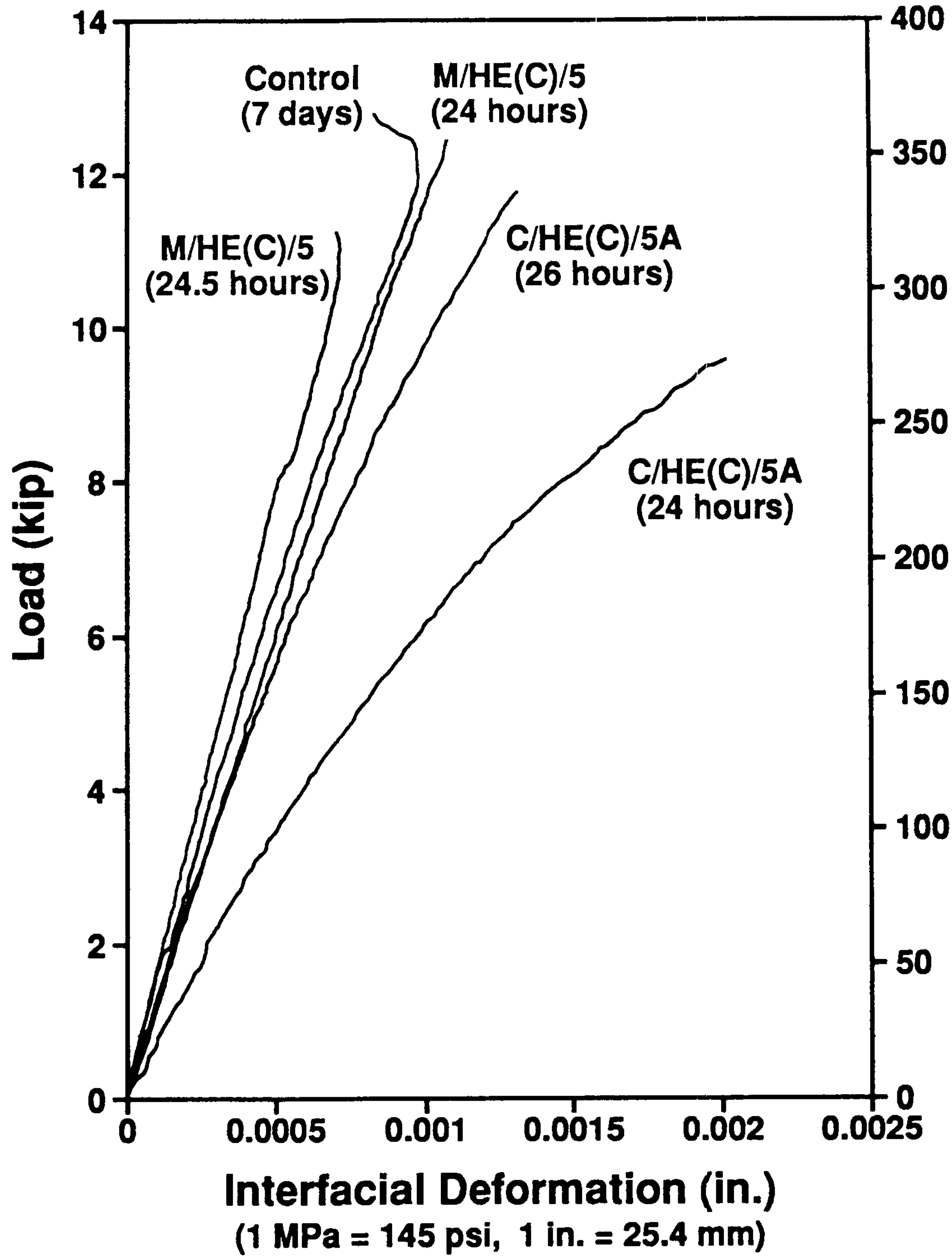

Figure 6.20 Load vs. interfacial deformation for HES C-C bond

6.8.2 Specimen Preparation

The C-S bond test specimens were cast with the #6 bar at 2.5 in. (63 mm) from the bottom of the concrete beam with a concrete cover of 3 in. (75 mm) on each side. Polyvinyl chloride (PVC) pipe sleeves with a 1 in. (25 mm) inner diameter surrounded the reinforcing steel bar on either end of the designed embedment length of 15 in. (375 mm). This setup minimized the local effect from the point loading. The embedment length used for the #6 bar was computed using the ACI Code equation (1993b) based on a design strength of 5,000 psi (35 MPa) for HES concrete and a steel yield strength of 60,000 psi (420 MPa).

6.8.3 Test Results and Discussions

The results of the C-S bond tests are reported in Table 6.13. The values of concrete strength at the design age of 24 hours are the average compressive strength of two 4 x 8 in. (100 x 200 mm) replicate specimens. Figure 6.21 shows the steel stress vs. net slip at the pulling end for two replicate specimens of HES concrete with CG. The bars reached a stress of 60,000 psi (420 MPa) at roughly 0.015 in. (0.37 mm). Therefore, the provisons of the ACI Code (1993b) for the development length are adequate in this case. Note that although the ACI Code equation was developed for concretes that reach their design strength at 28 days, the equation is also applicable for HES concrete.

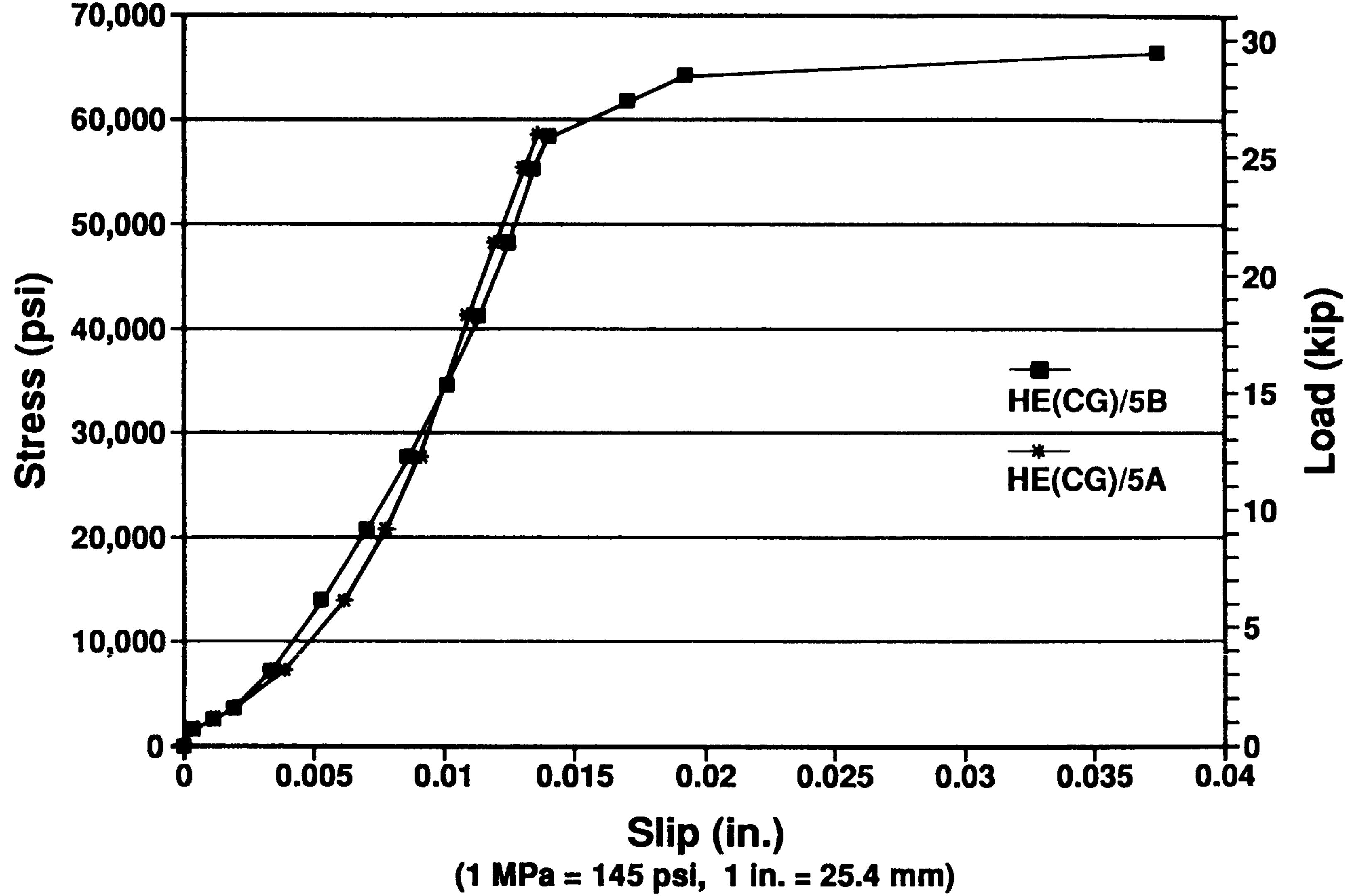

Figure 6.21 Stress vs. net slip for HES C-S bond test

7

Field Installation

7.1 Introduction

A major task of this research project was to construct a number of field installations in various parts of the country. Construction of the installations began in June 1991 and all were completed by July 1992. A total of five installations were constructed in New York, North Carolina, Illinois, Arkansas, and Nebraska.

Originally, field sites were scheduled only for North Carolina, Illinois, and Arkansas. The New York site was added due to the state's interest in this research project. An inquiry on the part of the research team ultimately led to the Nebraska installation. It had been reported that Nebraska was using a concrete with rapid strength development characteristics in its highway construction and repair. It was suggested that an installation be placed in Nebraska for a side-by-side comparison of the concrete developed by this project with that being used by the state's Department of Roads. Although it was not possible to construct sections side by side, the state did agree to provide a test site.

Except for the North Carolina installation, all sites involved the construction of full-depth and full-lane-width patches. The patch layout, materials and construction techniques used, and tests performed were similar at four locations. In contrast to the other locations, the North Carolina site involved new construction, made use of a variety of paste and aggregate combinations, and employed both conventional concrete and standard construction techniques (as control) and high performance concrete with a rapid construction schedule.

The five installations provide an opportunity to study the effects of a fairly wide range of exposure conditions. The New York and Illinois installations are in regions where a hard winter freeze is likely. The North Carolina site is in a mild marine environment. The potential for freezing-thawing cycles is high in Arkansas and very high in Nebraska. Traffic varies from light volume with occasional heavy loads in Nebraska to heavy volume with a high percentage of trucks in Arkansas.

Details of the construction and current findings (as of spring 1993) for each installation are presented in Sections 7.4 through 7.8. Due to the similarities among many of the sites, these sections are preceded by one that describes the layout and construction of a "typical" field site. Variations from the norm are highlighted as necessary in the sections on individual sites.

7.2 Objective and Scope

The primary objective of constructing the five field installations was to determine how various field service conditions might affect the mechanical properties and long-term performance of high performance concretes. Since these concretes were developed and tested in a laboratory environment, it was deemed important to evaluate them under field use conditions. In identifying potential sites for the installations, the following variables were considered: traffic volume, percentage of truck traffic, freezing-thawing potential, moisture potential, and possible use of de-icing agents. Availability and accessibility of sites were also considered in the selection process.

One of the criteria established early in this research was that the production of high performance concrete should be possible *using local materials and construction techniques*. Thus, a second objective of the field trials was to determine how easily high performance concrete could be adapted to production in the field. It was important to evaluate how readily this material could be batched, mixed, transported, placed, and finished. Therefore, local ready-mix producers and contractors were employed for the construction of the test sections. Also, each state's standard construction methods were generally used at each site. Highway department personnel were used to construct the Arkansas installation, as is typically the case in this state. A paver was used for the new construction in North Carolina.

The final objective was to identify the need for any changes to the mix, production process, or construction techniques. For example, it was found that the capacity of the scales in most batch plants may not be sufficient for the cement weights required with high performance concrete. Thus it may be necessary to split the weights of all materials and discharge one-half of the total at a time. Also, the research team wanted to identify any potential problems that may be encountered in the field. As a result, a list of "do's and don'ts" for the use of high performance concrete was developed.

Due to delays in finalizing the proportions for the very early strength (VES) mixture (as discussed in Chapter 4 of volume 2 of this report series), it was not possible to include this concrete in the construction of the field installations. However, in laboratory studies it was found that, when insulated, the high early strength (HES) mixture had strength development characteristics similar to those of the VES mixture. As a result, two patches were constructed at each location. One patch was insulated to mimic the VES mixture; the other was not insulated, as is standard for HES concrete. In North Carolina, an early version of the VES mixture was used.

7.3 Description of a Typical Field Installation

The physical layout of the site, materials and construction methods used, and monitoring program established were very similar for four of the five field installations. The North Carolina installation was unique in many respects. A description is provided below of what might be termed a "typical" site. Deviations from the norm are highlighted in subsequent sections on the individual field sites.

7.3.1 Site Description

The typical field installation involved the construction of two full-depth, full-lane-width patches. A single patch was roughly 45 ft (13.7 m) in length, 12 ft (3.7 m) wide, and 8 to 10 in. (200 to 250 mm) thick. Figure 7.1 depicts the layout of a typical patch. Transverse joints are spaced at approximately 15 foot (4.6 m) intervals. Thus, three 15 ft (4.6 m) sections make up the 45 ft (13.7 m) patch length.

7.3.2 Patch Description and Preparation

Concrete in the area to be patched was prepared for removal either the day before or on the morning the repair was made. The damaged pavement was removed by the lift-out method. Any subbase disturbed during removal of the old concrete was compacted using a hand-operated vibratory plate compactor.

Dowel bars were placed at all transverse joints. Holes were drilled into the existing concrete at the ends, and dowels were grouted in place. Dowel baskets were used at the internal joints, but no tie bars to adjacent lanes were used. All dowel bars were greased. Some states used welded wire mesh in constructing the patch. When used, the welded wire mesh was placed on chairs. A bond breaker was placed along the longitudinal joint.

Each patch was instrumented with a number of thermocouples in order to monitor temperature development during the first 24- to 48-hour period (see Figure 7.1). Type T constantan-copper thermocouples were used. A short length of small-diameter polyvinyl chloride (PVC) pipe was used to hold the thermocouples in position during placement of the concrete. The thermocouple wire was run along the subbase to the shoulder. All thermocouples were placed along the centerline of the patch. The outside sections of the patch had one thermocouple located at mid-depth. The center section was instrumented with three thermocouples. One was placed about 1 inch from the top of the patch, one at mid-depth, and the third about 1 inch above the subbase. Temperature readings were taken on a regular basis with a hand-held digital reader.

In addition to the two experimental patches, small full-lane-width and full-depth trial sections were constructed in Illinois, Arkansas, and Nebraska. These small patches allowed for last-minute adjustment of the mixture proportion with local materials, and they gave the batch plant and contractor an opportunity to work with the material. Also, cylinders could be taken from the mixture and used to confirm the strength of the concrete.

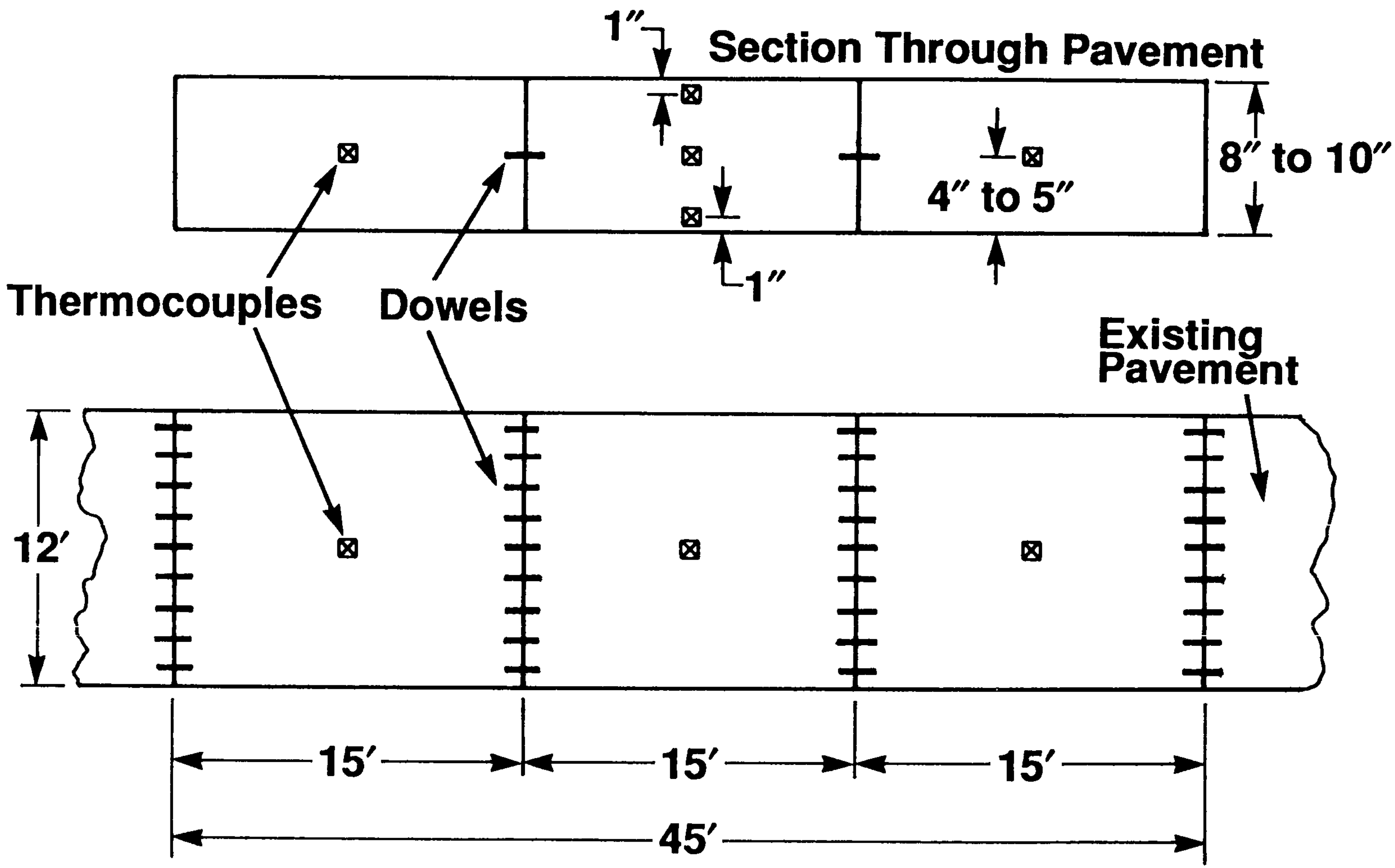

Figure 7.1 Typical field installation

7.3.3 Materials and Proportions

The HES mixture was used for each installation. Table 7.1 presents the mix proportions provided to each state prior to construction of the patches. These proportions are based on laboratory testing with a 3/4 in. (18.8 mm) maximum size limestone coarse aggregate. Although the maximum aggregate size varied somewhat, limestone was used at each site. The only admixture that was specially ordered for this work was the calcium nitrite accelerator/corrosion inhibitor. All other admixtures were those typically used by the ready-mix concrete producer.

Although the proportions in Table 7.1 call for 16 oz/cwt (10.44 mL/kg) of high-range water reducer (HRWR) and 3.5 oz/cwt (2.28 mL/kg) of air entraining agent (AEA), in most cases the actual dosage used in the field was slightly higher for both additives. Based on the experience gained in New York and North Carolina, the concrete used for the small trial sections in the other states was produced with 18 oz/cwt (11.74 mL/kg) of HRWR and 5 oz/cwt (3.26 mL/kg) of AEA. Based on the results from the trial sections, the admixture dosage rates were adjusted for the actual field installations. Generally, the first truck load of concrete at each site was produced with dosage rates of 18 oz/cwt (11.74 mL/kg) for the HRWR and 5 oz/cwt (3.26 mL/kg) for AEA.

7.3.4 Batching, Placing, and Curing

The concrete was dry-batched at the local ready-mix plant and transported to the job site. All materials, except the calcium nitrite, were added at the batch plant; the calcium nitrite was manually added at the job site. Due to the limited capacity of the scales at most batch plants, the weights for all materials had to be split. Thus, the batching process was modified slightly to accommodate the need to charge the truck twice for each load of concrete. Table 7.2 provides the basic batching sequence used. Each truck was charged with enough material, about 6 yd^3 (4.6 m^3), to fill one of the 15 ft (4.6 m) sections of a patch and to cast the necessary number of test specimens. This was done to simplify the testing program and future monitoring of the concrete.

When the truck arrived at the job site, the calcium nitrite solution was added to the concrete and mixed for about 30 revolutions. The truck was then brought into position and the concrete discharged by chute. After about 1 yd^3 of concrete had been discharged, a sample was obtained directly from the chute for use in determining the fresh concrete properties and casting the necessary test specimens. The remaining concrete in the truck was then discharged. On occasion, small amounts of water were added to the mix to improve workability. Only in New York were any chemical admixtures other than the calcium nitrite added at the job site. If the slump or air content was outside a desired range, adjustments in the admixture dosages were made at the batch plant. Radio communication with the batch plant made it possible to adjust any truckload of concrete.

After being discharged from the truck, the concrete was consolidated with a hand-operated vibrator immersed to the full depth of the slab. The concrete was manually struck off with a

Table 7.1 Basic HES mixture proportions

Material	Quantity
Cement (Type III)	870 pcy
Water*	299 pcy
Coarse aggregate	1,685 pcy
Fine aggregate	1,030 pcy
HRWR (naphthalene based)	15 oz/cwt
AEA (vinsol resin)	3.5 oz/cwt
Calcium nitrite	4.0 gal/cy

* Adjusted for free aggregate moisture and water in calcium nitrite. 1 pcy = 0.5933 kg/m^3, 1 gal = 3.78 L, 1 oz/cwt = 0.652 mL/kg.

Table 7.2 Basic batching sequence

1.	1/4 Water and 2/3 HRWR
2.	1/2 (Coarse and Fine Aggregate)
3.	1/2 Cement
4.	1/3 AEA
5.	1/2 (Coarse and Fine Aggregate)
6.	1/2 Cement
7.	2/3 AEA
8.	3/4 Water Less Wash Water
9.	1/3 HRWR
10.	Remaining Water

screed, floated, troweled, and textured with a wire broom. In some cases a vibrating screed was used. Because HES concrete has a tendency to be sticky, finishing was somewhat more difficult than normal. A liquid curing compound was then applied to the concrete.

In order to mimic the strength development characteristics of the VES mixture in the field, one of the two patches was insulated for 4 to 6 hours. The form of the insulation varied. Rigid polystyrene sheets, asphalt-based sheets, and blankets were used. Saw cuts were made at the transverse joints as soon as was practical. Saw cuts were not made on the insulated patch until the insulation was removed. Except for the Nebraska installation, all sites were kept closed overnight. Traffic was allowed on all patches by the following morning.

7.3.5 Testing

The slump, air content, unit weight, and as-placed temperature were determined, per AASHTO specifications, for each truckload of concrete brought on site. The concrete was sampled after approximately 1 yd^3 (0.8 m^3) had been discharged from the truck. Further placement of the concrete was slowed as the fresh properties were evaluated. The target range for slump was 3 to 5 in. (75 to 125 mm). The target range for air content was between 5% and 8%. On only one occasion was a truckload of concrete rejected based on its fresh concrete properties. This was in Illinois, where a very high air content resulted from a malfunction in the admixture dispensing equipment.

Thermocouples were placed in the patch area, as described in Section 7.3.2. In addition to these, a thermocouple was embedded in one of the 4 x 8-in. (100 x 200-mm) cylinders cast from the second truckload of concrete for each patch. These thermocouples were used to monitor the temperature history of the cylinders in the curing boxes. Readings in the cylinders were taken at the same time intervals as in the patch. These instrumented cylinders would be used to evaluate the curing conditions of the cylinders relative to the patch. Based on laboratory data, it was found that concrete placed in a curing box, as described below, will have a temperature history much like that at the mid-depth of a slab.

A large number of 4 x 8-in. (100 x 200-mm), and a few 6 x 12-in. (150 x 300-mm), cylinders were cast from each load of concrete. The center section of each patch was chosen to be the "representative" section for the patch, hence more specimens were cast from this concrete. Cylinders were cast for 1-, 7-, and 28-day testing from all sections of the patch. Cylinders were also cast for 6-, 12-, and 18-month testing from the center section. The center section of the insulated patch had still more specimens prepared for testing at 4, 5, 6, and 7 hours after placement. This was done in order to study the rate of strength gain and early-age properties of the mixture. A few 4 x 4 x 16-in. (100 x 100 x 400-mm) beams were cast from the concrete used for the center section of both the insulated and noninsulated patches.

The test cylinders and beams were cured for up to 1 day in a box constructed of extruded rigid foam insulation. For the cylinders, sheets of insulation were layered to a depth of 10 in. (250 mm). Holes were cut in the insulation to accept 4 x 8-in. (100 x 200-mm) cylinders. Two inches of insulation separated each hole, with 6 in. (150 mm) of insulation around the outside edge of the box. A schematic drawing of the box is in Figure 7.2. An early version of this box, used in New York and North Carolina, had only 2 in. (50 mm) of insulation around the outside. A similar type of box was built for the beam molds. These boxes had room for only two beams, primarily due to the weight of the molds and concrete. For the specimens taken from the insulated patch, a sheet of insulation was placed over the top of the curing box for as long as the slab was insulated. When the insulation was removed from the slab, the specimens in the curing box were uncovered but remained in the box. After curing in these boxes for 24 hours (except those tested earlier), the cylinders and beams were removed. The cylinders were left in their plastic molds, but the beams were removed from their steel molds. The cylinders and beams

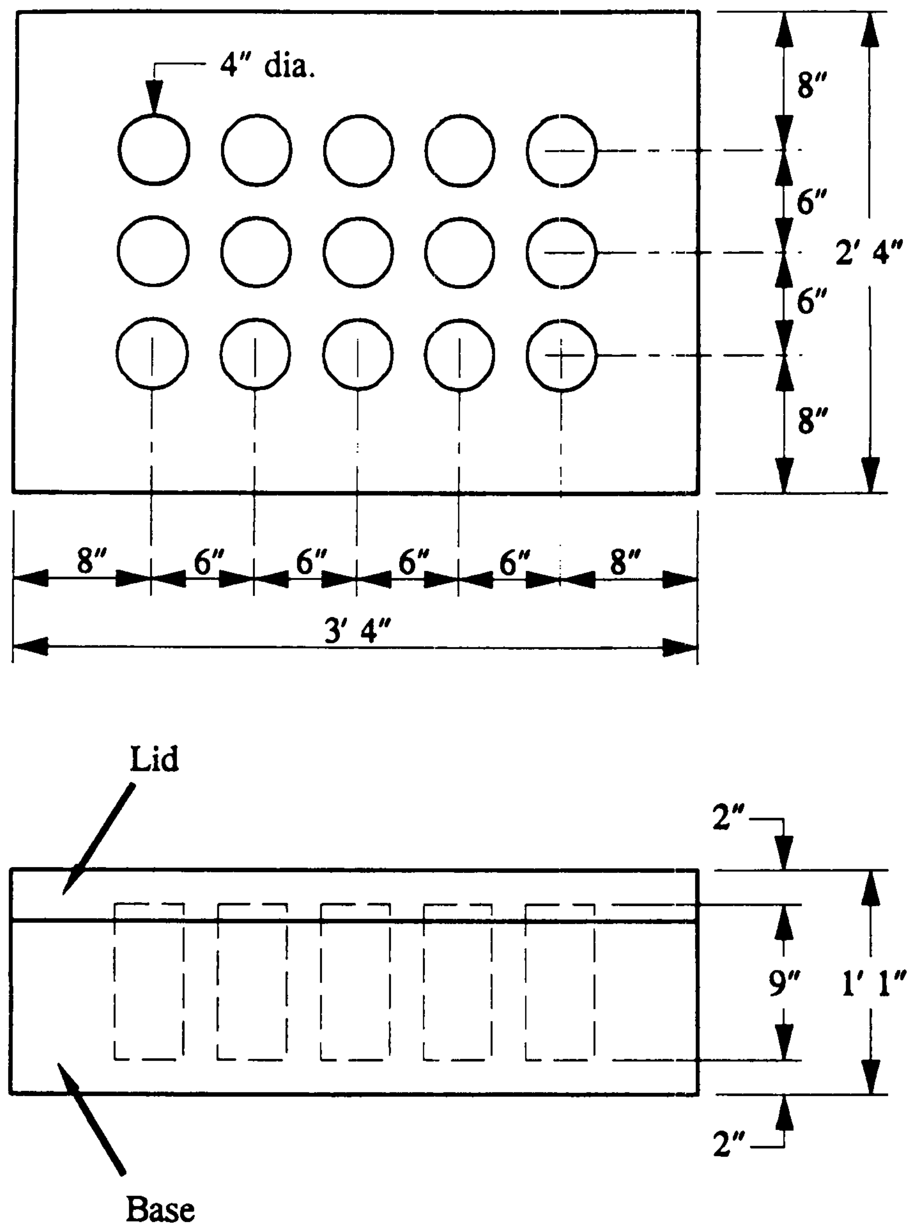

Figure 7.2 Field curing box

were buried along the side of the road with their top surfaces exposed to the environment. Highway department personnel retrieved and tested the specimens as scheduled.

A few 4 x 8-in. (100 x 200-mm) and 6 x 12-in. (150 x 300-mm) cylinders were cast and cured according to each state's standard procedures. These cylinders were tested at either 1 or 28 days of age.

The patches were also cored at 28 days and 6, 12, and 18 months for comparison with the cylinder strengths. At 28 days, cores were taken from outside the wheel path. At all later ages, cores were taken within the wheel path. Three cores were taken from the center section of each patch. One core was taken from all other sections.

Except for early-age cylinder tests, cylinders and cores were all tested according to each state's standard procedure. Arkansas and Illinois sulfur-capped their cylinders. Neoprene pads were used in Nebraska. In most cases, neoprene pads were provided by the research staff for early-age tests.

Visual inspections of the patches were to be made by state highway department personnel each time cylinders were retrieved for testing. In addition, a research staff member had planned to visit each site prior to the completion of the research project. By April 1993, the North Carolina, Illinois, Arkansas, and Nebraska sites had been visited, but not the New York site. However, the New York Department of Transportation (NYDOT) had not reported any problems.

7.4 Test Pavement — New York

7.4.1 Site Description

The first field installation was constructed on June 25, 1991. This installation is about 50 miles (81 km) west of Albany, New York on Interstate 88 near the town of Worcester in Otsego Country. The patch is located in the eastbound passing lane near reference marker 88I-9406-3158.

I-88 is a four-lane divided highway in a rural area. The traffic lanes are 12 ft (3.7 m) wide with inner and outer shoulders 4 and 10 ft (1.2 and 3 m) wide, respectively. Contraction joints are regularly spaced at 20 ft (6.1 m). Epoxy-coated dowels are used for load transfer. The original pavement is approximately 12 years old. The pavement is roughly 9 in. (225 mm) of plain portland cement concrete. Approximately 12 in. (300 mm) of sand, gravel, slag, and/or stone is located below the concrete. The shoulders are asphalt.

The New York installation was part of a repair/maintenance contract. Longitudinal joints were being resealed, and cracked slabs were being repaired. In the vicinity of the installation, the concrete was badly cracked. Away from the installation and other NYDOT repairs, the existing pavement appeared to be in good condition. There was evidence that when a nearby patch was placed, a significant compressive stress was relieved. This resulted in about a 1 1/2 in. (38 mm) movement of the pavement. Tie bars along the existing longitudinal joint had likely been sheared. The longitudinal joint appeared to have opened about 1/2 in. (13 mm).

The AADT at the installation is approximately 6,200 with about 20% trucks. The climatic exposure of the pavement can be described as wet with a hard freeze. De-icing salts have reportedly not been used since the installation was constructed.

7.4.2 Patch Description and Preparation

The New York installation consists of only one patch approximately 60 ft (18.3 m) long with doweled joints spaced at 20 ft (6.1 m) intervals. The patch is 12 ft (3.7 m) wide and 9 in. (225 mm) thick.

The concrete within the patch area was prepared for removal the day before the new concrete was to be placed. Traffic was permitted across the patch area overnight. The damaged pavement was removed the following morning. Although the subbase appeared firm upon removal of the old concrete, it was compacted. About 1/2 to 1 in. (13 to 25 mm) of sand was used to bring the subbase to the proper grade. Plastic sheeting was placed over the sand.

Epoxy-coated dowel bars were used at all joints. Dowels placed in the existing pavement had end caps to accommodate expansion. These dowel bars were not grouted. Dowels at the internal joints did not have end caps. A bituminous fiberboard 1/2 in. (13 mm) thick was used at all transverse joints. The patch was constructed using welded wire mesh. The longitudinal joint was greased as a bond breaker.

Thermocouples were placed in each section of the patch. The mid-depth thermocouple placed in the center section of the patch apparently was damaged during placement of the concrete and would not provide reliable data. All other thermocouples functioned properly.

7.4.3 Materials and Proportions

The concrete used for this installation was based on an early version of the HES mix. Table B.1 provides proportions for two mixes furnished to NYDOT for trial batching. The HES mix proportions were updated prior to construction of the New York installation. The state decided to use the updated mix proportions. Table B.2 gives the proportions used by NYDOT in laboratory trial batching of the HES mix. The coarse aggregate was a crushed limestone and all admixtures were W. R. Grace products. Table B.3 provides the fresh and hardened concrete properties for all trial batches. Batch 5 was selected by NYDOT for the field installation. The proportions of this mix are summarized in Table 7.3. Table 7.4 provides physical data on the New York aggregate, and Table 7.5 is adapted from a mill report on the Type III cement used for the project.

Table 7.3 Final mixture proportions — New York

Material	Mix 1
Cement (Type III), pcy	810
Water*, pcy	276
Coarse aggregate (SSD), pcy	1790
Fine aggregate (SSD), pcy	1040
HRWR (WRDA-19), oz/cwt	21
AEA (Daravair), oz/cwt	6
Calcium nitrite (DCI), gal	6

* Adjusted for free aggregate moisture and water in the calcium nitrite.
1 pcy = 0.5933 kg/m^3, 1 gal = 3.78 L, 1 oz/cwt = 0.652 mL/kg.

Table 7.4 Material properties and aggregate gradation — New York

Sieve Size	Percent Passing	
	Coarse	Fine
1"	100.0	
1/2"	98.2	
1/4"	14.4	
No. 4		94.0
No. 8		87.9
No. 16		78.5
No.30		50.3
No. 50		16.8
No. 100		5.6
No. 200		0.8
Specific Gravity	2.68	2.65
Absorption, %	0.6	0.8

1 in. = 25.4 mm

Table 7.5 Properties of Type III cement — New York

Chemical Analysis	Percent
Loss on ignition	1.3
Silicon dioxide (SiO_2)	20.3
Calcium oxide (CaO)	62.2
Aluminum oxide (Al_2O_3)	5.9
Iron oxide (Fe_2O_3)	2.2
Magnesium oxide (MgO)	2.8
Sulfur trioxide (SO_3)	3.9
Sodium oxide (Na_2O)	0.28
Potassium oxide (K_2O)	1.17
Insoluble residue	0.12
Ratio of Al_2O_3 to Fe_2O_3	2.7
Tricalcium aluminate (C_3A)	12
Total alkali oxides (Na_2O)	1.05
Water-soluble alkali	
Sodium Oxide (Na_2O)	0.16
Potassium Oxide (K_2O)	1.14
Total water-soluble alkali	0.91

7.4.4 Batching, Placement, and Curing

Based on the size of the patch and the number of test specimens that were to be cast, approximately 23 yd^3 (17.6 m^3) of concrete were required for the installation. Three truckloads of material were used. Trucks 1 and 2 each contained 7.5 yd^3 (5.7 m^3) of concrete; Truck 3 contained 8.5 yd^3 (6.5 m^3). The additional concrete in Truck 3 was to account for a possible shortfall due to the material used in sampling and testing.

A modified batching procedure was employed. Batching of the constituent materials was done manually and in two stages. Two factors led to the decision to batch in this manner. First, there was some concern over the potential for early setting of the concrete. Because workers have only limited experience with high performance concretes in the field, it was felt that there might not be enough time to place and finish the material with the nearest batch plant 30 minutes from the job site. Second, the batch plant did not have the facilities to store anything other than Type I cement in bulk form. Thus, the Type III cement required for the HES mix was delivered in bags.

Initially, each truck was charged with the coarse and fine aggregates at the batch plant. Prior to being batched, the moisture content of the aggregates was determined. The trucks then proceeded to a NYDOT maintenance substation a few miles from the installation. The cement, water, and admixtures were added to the trucks at the substation. The AEA was added first. This was accomplished using 5 gal (18.9 L) plastic buckets and manually pouring them into the truck. Next, the cement was added. A bucket and crane were used to charge the trucks with the cement. Sixty-five 94 lb. (42.6 kg) bags of cement were required per truck due to the high cement content of the concrete. Thus adding the cement took some time, as the concrete bucket used could hold only 15 to 18 bags of cement at a time. Water, adjusted for the moisture content of the aggregates and the water content of the calcium nitrite (DCI), was then added through a badger meter. Like the AEA, the HRWR was measured out in 5 gal (18.9 L) plastic buckets and manually poured into the truck. With everything but the DCI batched, the drum was turned for 70 revolutions. Finally, the DCI was added to the mix through a pulse meter. The drum was turned for an additional 30 revolutions.

Once all materials were added and mixed, the truck proceeded to the job site. Travel time from the substation was about 5 minutes. Due to the time involved in batching the cement, batching of successive trucks began immediately upon departure of a truck from the substation. The material quantities for each truck are given in Table 7.6.

No field trial batches were produced as part of this field installation. Thus, adjustment of the admixture quantities to achieve an acceptable slump and air content had to be done on the job.

Construction of the patch began on the northeast end and proceeded against the flow of traffic. The concrete from Truck 1 was placed in the northeast section of the patch, while the concrete from Truck 3 was placed in the southwest section.

Table 7.6 Actual batch weights — New York

Material	Truck 1	Truck 2	Truck 3
Batch Size, yd^3	7.5	7.5	8.5
Cement (Type III), lb	6,110	6,110	6,956
Water, lb	1,425	1,425	1,617
Coarse Aggregate, lb	13,350	13,320	14,090
Fine Aggregate, lb	8,080	8,130	9,420
HRWR (WRDA-19), gal	10	7.5	8.5
AEA (Daravair), oz	364.5	364.5	414
Calcium Nitrite (DCI), gal	45	45	51

1 lb/yd^3 = 0.5933 kg/m^3, 1 gal = 3.78 L

As will be discussed below, the slump and air content of the concrete in Truck 1 were well above acceptable levels. To compensate for this, only one-half of the original dosage of HRWR was used in Truck 2. The slump and air content for the Truck 2 were both fairly low when the truck arrived. One-half of the remaining HRWR was added to this truck for a total of three-fourths of the original dosage. Truck 3 used three-fourths of the original dosage of HRWR without alteration. The slump and air contents of Truck 3 were acceptable (see Table 7.7).

The concrete was discharged, consolidated, and finished as described in Section 7.3. The entire patch remained uncovered until concrete from the Truck 3 had set up. This was determined by touching the concrete. When no residue remained on the hand, the concrete was considered to have set. This occurred a few hours after batching of the concrete from Truck 3. A polyethylene sheet was then placed over the entire patch. Closed-cell extruded polystyrene insulation board, 2 in. (50 mm) thick, was immediately placed over the plastic sheet and held in place with sandbags.

NYDOT practice is to open a patch to traffic after the concrete has reached 2,000 psi (14 MPa) compressive strength. NYDOT has found, with its calcium chloride patch mix, that this strength develops within about 6 hours after placement. The same criteria were applied to the field installation. Once the concrete reached 2,000 psi (14 MPa), the insulation was to be removed, saw cuts made, and a curing compound applied. After 7 hours (8 p.m.), cylinders cast from Truck 2 had exceeded 2,000 psi (14 MPa), while cylinders from Truck 1 were just over 1,000 psi (7 MPa) (see Section 7.4.5). Therefore, it was decided that the concrete would remain insulated overnight and the patch not opened to traffic until the following morning.

The following morning (6/26/91) the insulation was removed, saw cuts were made, a curing compound applied, and the patch was opened to traffic. No evidence of cracking was reported at that time. Sawing of the joints was apparently completed without difficulty.

Table 7.7 Fresh concrete properties — New York

Truck and Time	Slump (in.)	Air (%)	Unit Weight (lb/ft^3)
Truck 1			
12:20 p.m.	8.75	12.0	133.55
12:30 p.m.	8.25		
1 p.m.	7.25	10.3	138.15
Truck 2			
1:20 p.m.	2.5	4.8	146.81
1:35 p.m.*	7.5	8.2	141.09
Truck 3			
2:15 p.m.	6.0	8.0	142.14

* Measured after second addition of HRWR.
1 in. = 25.4 mm, 1 lb/ft^3 = 16 kg/m^3

7.4.5 Results and Discussions

All fresh concrete testing was conducted by NYDOT personnel. The research staff cast the cylinders. No beams were cast in New York.

Because the New York installation was the first for the SHRP project and personnel was limited, the decision was made to focus all efforts on casting test specimens from Truck 2. Only two cylinders were cast for any test age. A few cylinders were also taken from Truck 1 due to the high air content of that concrete. No cylinders were cast from Truck 3.

Cylinders cast for early-strength testing were transported to the NYDOT materials laboratory in Albany as soon as they could be moved without damage. These cylinders remained in their curing box until tested.

In addition to the 4 x 8-in. (100 x 200-mm) cylinders, two 6 x 12-in. (150 x 300-mm) cylinders were cast from the concrete in Truck 2. These cylinders were insulated and tested in the Albany laboratory at 1 day. These cylinders were cast for comparison with the 4 x 8-in. (100 x 200-mm) cylinders.

7.4.5.1 Fresh Concrete Properties

Table 7.7 lists the fresh concrete properties for all three truckloads of concrete. Multiple tests were conducted on the concrete from Trucks 1 and 2.

The initial slump and air content of the concrete in Truck 1 were very high, for reasons that are not entirely clear. It may be that the batching sequence, timing, and mixing action experienced in the field was different enough to cause the fresh concrete properties to vary noticeably from those obtained in the laboratory. A second slump test was performed to verify the results of the first test. Again, the slump was high. Due to the high slump and air content, placement of concrete from Truck 1 was stopped. In an effort to decrease the air content and permit the concrete to stiffen, the drum was allowed to agitate for about 15 more minutes. The concrete was then sampled a second time. The slump and air content remained high at 7.25 in. (181 mm) and 10.3%, respectively. Regardless, the remaining concrete was discharged into the patch as no excess materials were available for this installation. At that point, NYDOT officials decided that test cylinders should be cast using concrete from Truck 1.

Based on the properties of the concrete from Truck 1, the initial dosage of HRWR was cut in half for Truck 2. The amount of AEA was not changed. The initial slump and air content were now somewhat low. The concrete was very stiff and hard to work. As a result, one-half of the remaining HRWR was added to the truck at the job site. The resulting dosage of HRWR was three-fourths of that used for Truck 1 (see Table 7.6). The concrete was allowed to mix for a few minutes. Another sample was taken and tested. Both the slump and air content had increased significantly. The remaining material was discharged from the truck. Cylinders were cast from this concrete after the second addition of HRWR.

The concrete from Truck 3 was batched with three-fourths of the originally intended dosage of HRWR. The dosage rate of AEA was the same as for Trucks 1 and 2. The slump and air contents were higher than desired but were deemed acceptable. This concrete was discharged without modification.

It can be seen from Figure 7.3 that the temperature history of the cylinders in the curing box corresponds very well with the concrete in the patch up to 6 hours after placement. Recall that the mid-depth thermocouple for the concrete from Truck 2 malfunctioned and no data were actually recorded for this point. However, the cylinder temperature follows what might have been expected for the mid-depth of the slab.

7.4.5.2 Hardened Concrete Properties

Initially, cylinders for strength testing were to be taken only from Truck 2. Due to the high slump and air content of the concrete from Truck 1, three 4 x 8-in. (100 x 200-mm) cylinders were cast for early-age strength testing. No cylinders were cast from the concrete from Truck 3. Results from all compression tests are presented in Tables 7.8 and 7.9.

The three cylinders cast from the concrete from Truck 1 were tested at 4, 8, and 20 hours after placement, respectively. The first cylinder had not adequately set up in 4 hours and therefore was damaged on removal from its mold. The other two cylinders were tested as scheduled. The low strength, 1,100 psi (7.6 MPa), of the cylinder tested at 8 hours resulted in the entire patch being kept closed to traffic until the following morning. Had this cylinder reached a strength of 2,000 psi (13.8 MPa) or more, the patch would have been opened the evening of the placement.

Twelve 4 x 8-in. (100 x 200-mm) cylinders were cast from the concrete from Truck 2 for early-age testing. Three cylinders were tested at 1 hour intervals beginning at 4 hours after placement. One of the cylinders tested at 4 hours was damaged on removal from its mold. Two 4 x 8-in. (100 x 200-mm) cylinders were also cast for laboratory curing and testing at 1 day. These cylinders were actually tested at about 20 hours and are believed to have been insulated and not laboratory-cured. As can be seen from the data in Table 7.8, the concrete from Truck 2 had reached more than half of its target 24-hour strength in just 7 hours. The cylinders tested at 20 hours exceeded the target by 1,300 psi (9.0 MPa). All cylinders were sulfur-capped and tested on a 300 kip (136,000 kg) Tinius Olsen machine.

An additional 13 cylinders were cast and cured in a separate box for long-term testing. Of the 13 cylinders, one was used for monitoring the internal temperature of the cylinders in the box. The remaining 12 were used for compression testing. Two cylinders were retrieved from the job site and tested at 1, 7, and 28 days and also 6, 12, and 18 months. Except for the 1-day test cylinders, which were brought to the Albany laboratory by the research staff, the cylinders were retrieved by NYDOT personnel. The results of the long-term testing are presented in Table 7.9.

Table 7.9 shows that the concrete met its target strength of 5,000 psi (35 MPa) in 24 hours. The variation in strength among cylinders is somewhat more than expected based on previous

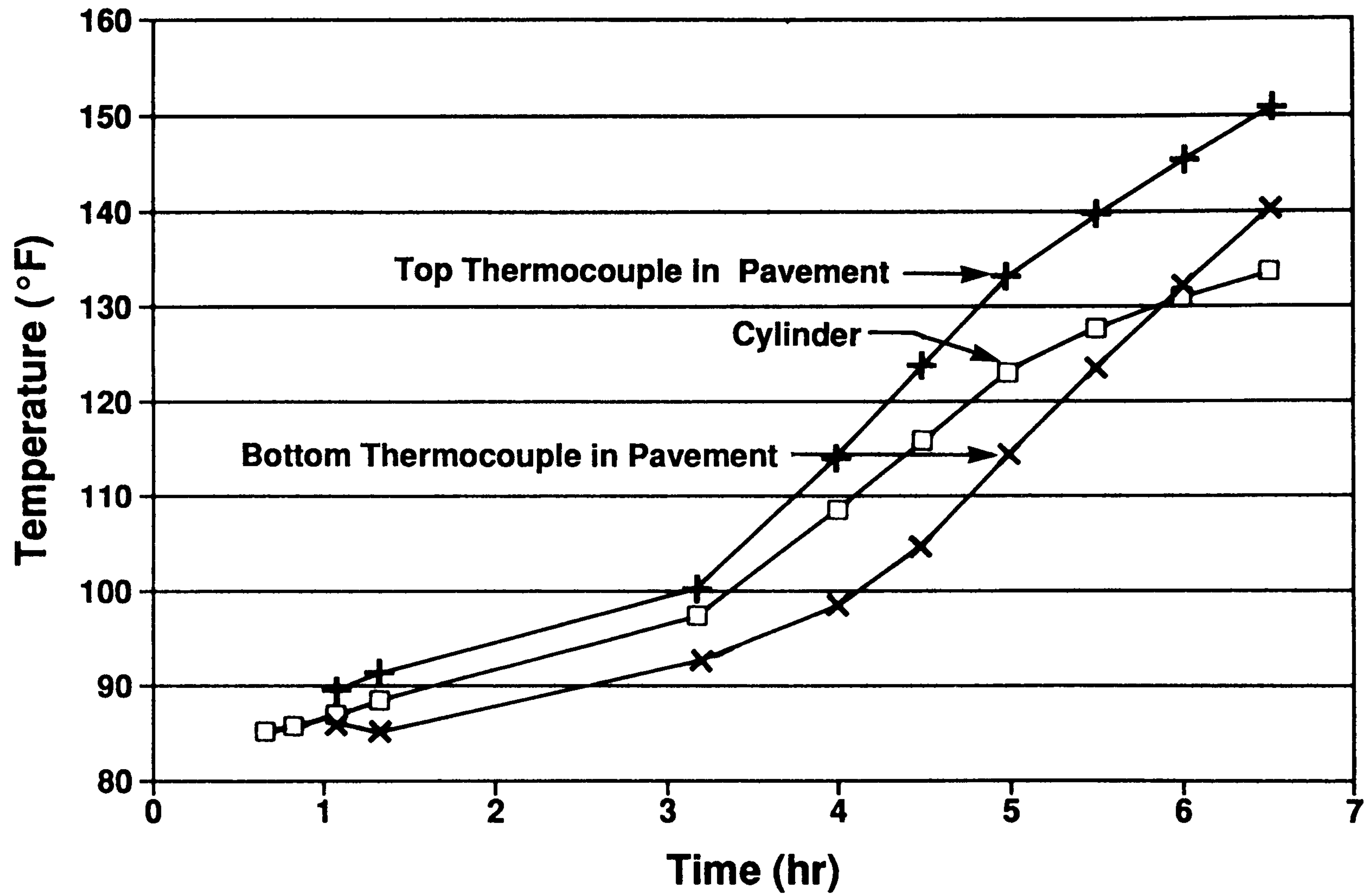

Figure 7.3 Truck 2 concrete temperature data (insulated) — New York

Table 7.8 Early strength (psi) of 4 x 8-in. cylinders — New York

Test Date	Test Age	Truck 1	Truck 2
6/25/91	4 Hours	Not Set	Not Set
			160
			160
	5 Hours		450
			580
			580
	6 Hours		1,550
			1,450
			1,540
	7 Hours		2,650
			2,660
			2,780
	8 Hours		1,100
6/26/91	20 Hours	4,550	6,370*
			6,300

*Believed to have been laboratory-cured. 145 psi = 1 MPa, 4 x 8 in. = 100 x 200 mm

Table 7.9 Strength (psi) of 4 x 8-in. cylinders —New York

Test Date	Test Age	Truck 2
6/26/91	1 Day	5,310
		6,730
		5,840*
		5,820*
7/2/91	7 Days	7,450
		7,610
7/23/91	28 Days	6,650
		7,590
12/2/91	6 Months	8,330
		7,880
6/25/92	12 Months	9,140
		9,010
12/921	18 Months	8,590
		8,910
		9,070*
		9,010*

* 6 x 12 in laboratory-cured cylinder. 145 psi = 1 MPa, 4 x 8 in. = 102 x 203 mm, 6 x 12 in. = 150 x 300 mm

laboratory testing. This may be due in part to inexperience with the concrete as well as the casting and testing of only two cylinders for each test date. Also included in this table are the results from testing two 6 x 12-in. (150 x 300-mm) cylinders cast by NYDOT. These cylinders were insulated in rubber foam insulation and transported to the NYDOT laboratory for testing at 1 day. The strengths of these cylinders were comparable to those of the field-cured 4 x 8-in. (100 x 200-mm) cylinders tested at the same time. The strength of the concrete appeared to level off after about 7 days. Strength variation among cylinders is high at 28 days and 6 months. Thus it is difficult to speculate as to the actual strength development characteristics of this concrete.

Table 7.10 presents the results for nominal 4-in. (100-mm) cores taken from the patch. The cores were sawed just above the welded wire fabric resulting in test specimens shorter than the slab thickness of 9 in. (225 mm). The results indicate that the concrete from Truck 1 remains weaker than that from the other trucks even after 1 year. The concrete from Truck 3 is stronger than that of Truck 1 but not as strong as that from Truck 2. The additional strength of the concrete from Truck 2 may be due to the delayed addition of some of the HRWR. The core data follow the cylinder data reasonably well. The wide range of variation between specimens does not occur for the cores as it did for the cylinders.

Table 7.10 Strength (psi) of 4-in. cores — New York

Test Date	Test Age	Truck 1	Truck 2	Truck 3
7/23/91	28 Days	5,460	7,040 7,070 7,160	6,350
12/2/91	6 Months	6,330	7,290 7,880 8,080	6,930
6/25/92	12 Months	6,060	8,200 7,830 7,840	7,220
12/92	18 Months	6,240	8,900 8,420 6,480	7,080

145 psi = 1 MPa, 4 in. = 100 mm

7.4.6 Follow-up Site Visit

As of April 1993, the New York installation was 22 months old. No inspection had been made by the research staff, and no visit is planned due to time limitations. However, reports from NYDOT indicate that the patch is still in good condition.

7.5 Test Pavement — North Carolina

7.5.1 Site Description

In early 1991, a two-way, two-lane bridge was being replaced by two larger two-lane, eventually one-way, high-rise bridges. The bridges were located on US 17 over the Roanoke River, just north of Williamston, North Carolina, approximately 25 miles (40 km) from Albemarle Sound in the northeastern part of the state. One bridge, which would eventually serve northbound traffic, had already been constructed and was in service providing two-way traffic until the other (southbound) bridge was completed.

Originally, a short concrete approach slab was to be constructed in July 1991, which would tie in to the new asphalt road leading to the bridge. The plans were modified to provide a two-lane experimental placement for the field installation, approximately 180 ft (55 m) long (see Figure 7.4). This experimental section would abut an approach slab constructed of conventional concrete and would then tie in to the new asphalt road. The grade for the bridge approach was sloping upward toward the bridge, to the south. The two lanes were not in service, except for construction traffic. The north end of the bridge is at station 51+50, the beginning of the high performance concrete is at 52+22, and the end of the high performance concrete is at 54+02. Between 54+02 and 54+47 is conventional concrete. The asphalt pavement begins at 54+47.

Weather during the construction period was very hot and humid with frequent showers. Construction was begun as early as possible in the day to minimize working in the high temperatures of midday and afternoon. Temperatures at the beginning of the day were typically in the mid-70s with afternoon highs in the low to mid-90s. Therefore, ambient temperatures were generally in the mid- to upper 70s at batch time for much of the work.

7.5.2 Pavement Description and Preparation

The concrete pavement was unreinforced, jointed at 15 ft (4.6 m) intervals, and doweled at the end of each day's placement with a maximum of 120 ft (36.6 m) between doweled sections. The concrete was placed on an asphalt base course or "black-base." Depth of the pavement varied somewhat to provide a smooth transition, with a minimum of 9 in. (225 mm).

The two lanes presented an opportunity to examine construction under two conditions. In the first series of placements on the inside or passing lane, concrete was more tightly controlled on a

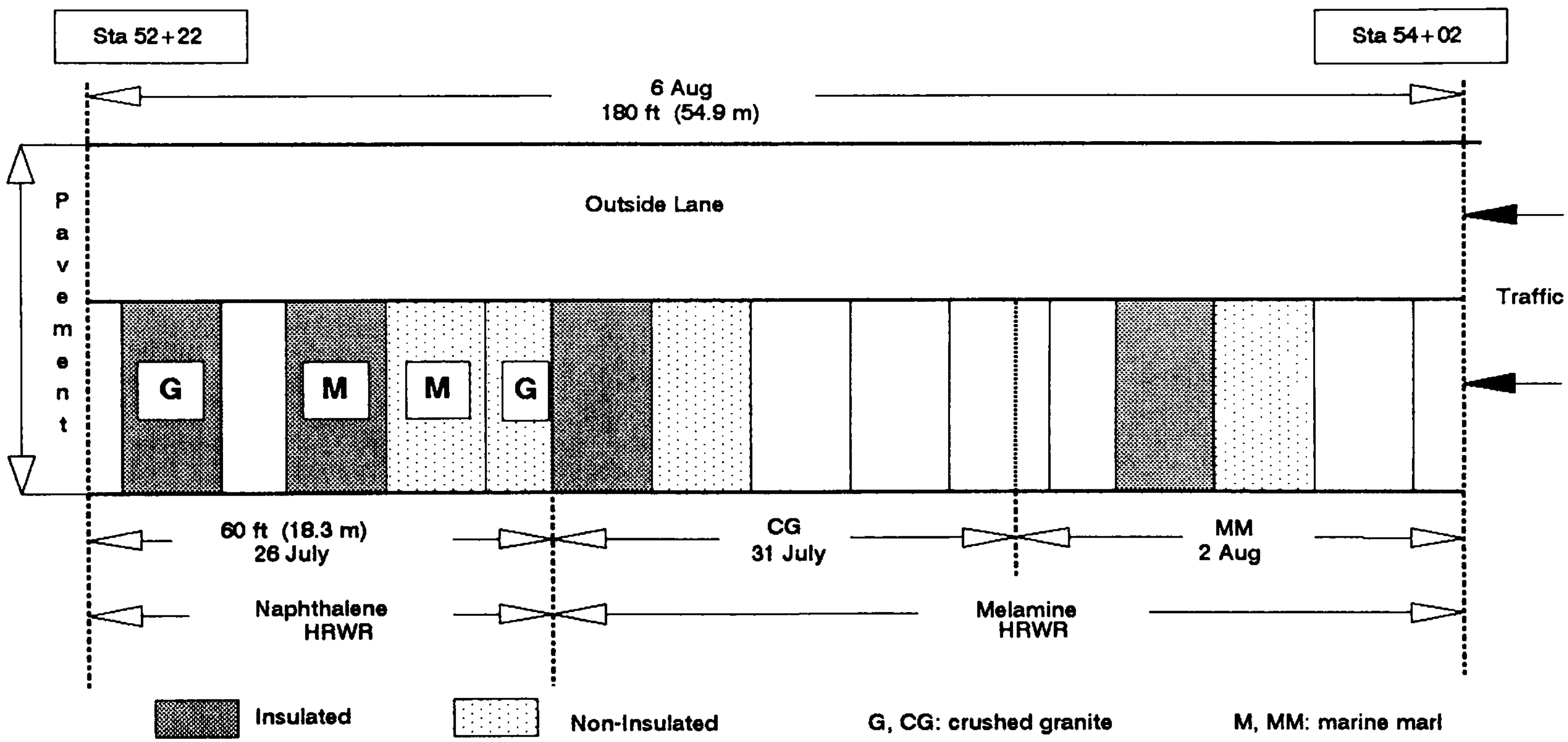

Figure 7.4 Pavement layout at Williamston, North Carolina

batch-to-batch basis, and emphasis was on testing and materials. Three different 60 ft (18.3 m) sections, one lane wide, were placed by this method. Each section was placed in 1 day.

In the second series of placements, conducted in the outside lane, typical batching and placement rates were used so that all 180 ft (54.9 m) were placed in 1 day. Here the emphasis was on the impact of routine construction methods on variations in product delivered. Due to weather delays, this lane was placed the week after the first series of three 60 ft (18.3 m) sections.

7.5.3 Materials and Proportions

The concrete used at the Williamston site was based on the HES mixture proportions, although two different types of HRWR were used. In some sections, a naphthalene-based HRWR was used; in others, a melamine-based HRWR was used. The melamine HRWR was produced by Cormix; the naphthalene HRWR was produced by W. R. Grace. Both types of HES mixtures were used in insulated and noninsulated sections.

A 30% solution of calcium nitrite, $Ca(NO_2)_2$, was added to all the high performance concrete used at this site. The calcium nitrite solution used is produced by W. R. Grace under the trade name Darex Corrosion Inhibitor™ (DCI). This product is intended to inhibit corrosion of reinforcing steel, but is also a fast-acting accelerator. It was therefore selected as a nonchloride accelerator that would also improve long-term durability of reinforced structures such as bridge decks, for which the HES mixture was primarily intended.

The only other admixture used in the HPC mixtures in the North Carolina field installation was a proprietary AEA manufactured by Master Builders, Inc., under the trade name Micro-Air.

Each type of mixture used two types of coarse aggregate: a crushed granite (CG) very similar to that used in the laboratory, and a marine marl (MM) from the same source as that used in the laboratory. Both met ASTM C 33 size #57 specifications, with virtually all the material passing the 1 in. (25 mm) sieve. The CG was a hard, angular aggregate of low absorption (0.4%). The specific gravity, saturated surface dry (SSD), was 2.64. The MM was a cubical to subangular, relatively porous, high-absorption (typically more than 4.5%, but variable) shell limestone. The specific gravity (SSD) of the MM was typically 2.48. Fine aggregate was the same in all mixes and was similar to that used in the laboratory.

The cement used was a Type III portland cement with a blaine fineness of about 5,500 and more than 0.6% alkalis as sodium oxide (see Table 7.11). The alkali content was about twice as high as that of the portland cement used in the previous laboratory investigations at North Carolina State University.

Nominal batch weights for the concrete mixtures used in the North Carolina installation are given in Table 7.12, with actual batch quantities given in Tables 7.13 through 7.17. All mixtures contained the same nominal cement content and had approximately the same water cement ratio (W/C). Mix designations were composed of three letters. The first letter designated the type of

HRWR, **H** for naphthalene-based HRWR or **V** for melamine-based HRWR. The second letter designated the type of coarse aggregate, **C** for crushed granite and **M** for marine marl. The third letter designated whether insulation was used on the slabs for the first 6 hours, that is, **I** for an insulated section or **N** for a non-insulated section. "HCI" would therefore indicate a mixture using a naphthalene type HRWR and crushed granite which was insulated during initial curing.

Table 7.11 Type III cement properties — North Carolina

Source: Roanoke Cement Co. (A division of Tarmac - Lonestar), Roanoke, Virginia. Results of grab sample tested by North Carolina Department of Transportation.

Physical Characteristics:

Blaine fineness:	518 m^2/kg
Initial set (Gilmore):	2 Hours 30 Minutes
Mortar cube strength:	
1 Day:	3,130 psi
3 Days:	4,930 psi

Chemical Analysis (%)		Calculated Compounds (%)	
CaO:	62.5	C_3S:	57.9
SiO_2:	19.30	C_2S:	11.7
Al_2O_3:	5.20	C_3A:	10.0
Fe_2O_3:	2.26	C_4AF:	6.9
SO_3:	4.10		
LOI:	1.29		

Total alkalis as Na_2O: 6.20%

Table 7.12 Nominal batch weight data — North Carolina

Material	Weight
Cement	870 pcy (516 kg/m^3)
Added Water	270 pcy (160 kg/m^3)
DCI	4 gal/cy (20 L/m^3)
Total Water	300 pcy (178 kg/m^3)

(Total water includes that contributed by DCI, AEA, and HRWR)

AEA: dosage as necessary, but as high as 2.5 oz/cwt (1.63 mL/kg) was required.

HRWR: up to 24 oz/cwt (15.7 mL/kg) of naphthalene HRWR or 28 oz/cwt (18.3 mL/kg) melamine HRWR; much higher dosages tended to produce unacceptable strengths at 6 hours due to excessive retardation.

Coarse aggregate (saturated surface dry):

Crushed Granite (CG)	Marine Marl (MM)
1,720 pcy (1,020 kg/m^3)	1,640 pcy (970 kg/m^3)

Sand quantities were adjusted as necessary to provide a yield of 1 m^3 (1 m^3 = 1.31 yd^3) at minimum air content.

Table 7.13 Plastic properties and batch quantities of concrete, all test sections, July 29, 1991, North Carolina

Test Section ID	HCN	HCI	HCN	HMI	HMN	HCN
Aggregate type	CG	CG	CG	MM	MM	CG
Insulation	No	Yes	No	Yes	No	No
Batch quantity (yd^3)	6	6	4	6	4	4
Slump (in.)	2.5	3.5	4.5	2.25	2.25	6
Air (%)	4.2	6.1	5.0	5.5	6.8	4.9
Unit weight (lb/ft^3)	143.5	143.4	143.5	139.4	136.7	143.5
W/C	0.34	0.34	0.32	0.33	0.33	0.34
Material quantity per batch:*						
Cement (Type III) (lb)	5,230	5,230	3,480	5,260	3,490	3,485
Sand (lb)	6,600	6,615	4,460	6,420	4,275	4,425
Moisture (%)	9.6	9.6	9.6	9.6	9.6	9.6
Coarse aggregate (lb)	1,046	1,052	621	866	567	695
Moisture (%)	0.4	0.4	0.4	7.4	7.4	0.4
Water (lb)	1,046	1,052	621	866	567	695
HRWR (naphthalene) (gal)	6.5	8.5	5.5	8.5	6	6
AEA (MicroAir) (oz/cy)	107	116	78	116	80	79
Notes:	Rejected: Slump		Wrong Aggregate			

*Includes 4 gal/yd^3 of 30% calcium nitrite solution (DCI) in all mixes.
1 yd^3 = 0.77 m^3, 1 in. = 25 mm, 1 lb/ft^3 = 16 kg/m^3, 1 lb = 0.45 kg, 1 gal = 3.78 L, 100 oz/cy = 3.87 L/m^3

Table 7.14 Plastic properties and batch quantities of concrete, all test sections, July 31, 1991, North Carolina

Test Section ID	VCI	VCN	VCN	VCN	VCN
Aggregate type	CG	CG	CG	CG	CG
Insulation	Yes	No	No	No	No
Batch quantity (yd^3)	4	6	6	4	5
Slump (in.)	3	3.5	3	2.25	2.5
Air (%)	6.2	5.7	4.8	4.5	4.2
Unit weight (lb/ft^3)	143.4	142.2	146.1	146.1	146.1
W/C	0.34	0.35	0.35	0.34	0.35
Material quantity per batch:*					
Cement (Type III) (lb)	3,485	5,220	5,225	3,490	4,370
Sand (lb)	4,335	6,525	6,510	4,350	5,415
Moisture (%)	7.9	7.9	7.9	7.9	7.9
Coarse aggregate (lb)	6,930	10,425	10,590	6,915	8,670
Moisture (%)	1.1	1.1	1.1	1.1	1.1
Water (lb)	733	1,125	1,125	750	941
HRWR (melamine) (gal)	6.5	10.5	10	6.75	8.75
AEA (MicroAir) (oz/cy)	80	120	120	80	106

*Includes 4 gal/yd^3 of 30% calcium nitrite solution (DCI) in all mixes.
1 yd^3 = 0.77 m^3, 1 in. = 25 mm, 1 lb/ft^3 = 16 kg/m^3, 1 lb = 0.45 kg, 1 gal = 3.78 L, 100 oz/cy = 3.87 L/m^3

Table 7.15 Plastic properties and batch quantities of concrete, all test sections, August 2, 1991, North Carolina

Test Section ID	VMN	VMN	VMI	VMN	VMN	VMN
Aggregate type	MM	MM	MM	MM	MM	MM
Insulation	No	No	Yes	No	No	No
Batch quantity (yd^3)	4	4	6	6	4	4
Slump (in.)	1	2	3	1.75	3	2
Air (%)	5.5	7.4	8	6.4	9.2	6.8
Unit weight (lb/ft^3)			143.3	135.1		
W/C	0.35	0.34	0.35	0.35	0.35	0.34
Material quantity per batch:*						
Cement (Type III) (lb)	3,490	3,505	5,245	5,220	3,485	3,495
Sand (lb)	4,275	4,275	6,525	6,375	4,260	4,275
Moisture (%)	9.5	9.5	9.5	9.5	9.5	9.5
Coarse aggregate (lb)	6,450	6,470	9,730	9,680	6,540	6,450
Moisture (%)	8.4	8.4	8.4	8.4	8.4	8.4
Water (lb)	567	567	850	850	567	567
HRWR (melamine) (gal)	7	8	14	14	10	10
AEA (MicroAir) (oz/cy)	80	88	132	132	88	88
Notes:	Rejected: Slump		Excellent Workability			

*Includes 4 gal/yd^3 of 30% calcium nitrite solution (DCI) in all mixes.
1 yd^3 = 0.77 m^3, 1 in. = 25 mm, 1 lb/ft^3 = 16 kg/m^3, 1 lb = 0.45 kg, 1 gal = 3.78 L, 100 oz/cy = 3.87 L/m^3

Table 7.16 **Plastic properties and batch quantities of concrete, MM sections, August 6, 1991, North Carolina**

Test Section ID	VMN	VMN	VMN	VMN	VMN	VMN	VMN
Aggregate type	MM	MM	MM	MM	MM	MM	MM
Insulation	No	No	No	No	No	No	No
Batch quantity (yd^3)	5	5	5	5	5	5	5
Slump (in.)	5	6.25	5.5	5.5	4.75	9.25	4.5
Air (%)	8.5	10.3	9.8	8.5	9.2	4.4	8.7
W/C	0.32	0.32	0.32	0.32	0.31	0.32	0.30
Material quantity per batch:*							
Cement (Type III) (lb)	4,385	4,355	4,370	4,375	4,340	4,385	4,350
Sand (lb)	5,175	5,145	5,175	5,175	5,175	5,175	5,175
Moisture (%)	6.1	6.1	6.1	6.1	6.1	6.1	6.1
Coarse aggregate (lb)	7,890	7,965	7,965	7,935	7,935	7,935	8,035
Moisture (%)	5.6	5.6	5.6	5.6	5.6	5.6	5.6
Water (lb)	966	958	958	958	916	958	875
HRWR (melamine) (gal)	8.5	9.5	9.5	9.5	9	9.25	9.25
AEA (MicroAir) (oz/cy)	110	115	115	110	110	108	105

*Includes 4 gal/yd^3 of 30% calcium nitrite solution (DCI) in all mixes.
1 yd^3 = 0.77 m^3, 1 in. = 25 mm, 1 lb/ft^3 = 16 kg/m^3, 1 lb = 0.45 kg, 1 gal = 3.78 L, 100 oz/cy = 3.87 L/m^3

Table 7.17 Plastic properties and batch quantities of concrete, CG sections, August 6, 1991, North Carolina

Test Section ID	VGN	VGN	VGN	VGN	VGN	VGN	VGN	VGN	VGN
Aggregate type	CG	CG	CG	CG	CG	CG	CG	CG	CG
Insulation	No	No	No	No	No	No	No	No	No
Batch quantity (yd^3)	5	5	6	6	2.5	5	5	5	5
Slump (in.)	4	3	0.5	0.5	2	6.25	5.75	6	7
Air (%)	6.8	5.4	3.4	4.2	5	6.4	7.8	7.5	7.4
W/C	0.33	0.33	0.31	0.31	0.34	0.35	0.33	0.34	0.35
Material quantity per batch:*									
Cement (Type III) (lb)	4,365	4,365	5,230	5,220	2,190	4,365	4,350	4,385	4,440
Sand (lb)	5,325	5,325	6,375	6,375	2,700	5,250	5,250	5,250	5,350
Moisture (%)	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1
Coarse aggregate (lb)	8,600	8,655	10,425	10,425	4,310	8,625	8,625	8,625	8,525
Moisture (%)	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Water (lb)	983	981	1,066	1,058	517	1,083	1,000	1,041	1,083
HRWR (melamine) (gal)	8.5	9	12	11.5	5	9.5	9.5	9.5	9.5
AEA (MicroAir) (oz/cy)	105	105	127	127	53	106	106	106	106

*Includes 4 gal/yd^3 of 30% calcium nitrite solution (DCI) in all mixes.
1 yd^3 = 0.77 m^3, 1 in. = 25 mm, 1 lb/ft^3 = 16 kg/m^3, 1 lb = 0.45 kg, 1 gal = 3.78 L, 100 oz/cy = 3.87 L/m^3

7.5.4 Batching, Placing, and Curing

The ready-mix concrete supplier operated out of a small dry-batch plant in Williamston, North Carolina. The plant did not have automated moisture control. Moisture determination and batch corrections were typically the responsibility of the North Carolina Department of Transportation (NCDOT) plant inspector when placing NCDOT concrete. Due to the relative lack of water in the mixture, this responsibility was handled jointly by the NCDOT inspector and the research staff at the batch plant. The lack of automated moisture control and the sensitivity of the mixture to variations in water content made it difficult to maintain the desired level of quality control at the batch plant. Due to scale capacity limitations, aggregate and cement were usually weighed out in two equal half-batches. For this reason and because the HRWR was added by hand, batching time was generally just under 10 minutes.

The batching sequence was modified slightly from conventional practice. In order to avoid head pack, it was necessary to add both water and HRWR to the first part of the load, which included portions of both fine and coarse aggregate. The specific batch sequence used for the North Carolina field installation was to first load one-third of the batch water and two-thirds of the HRWR into the truck. Then, with the drum rotating at mix speed, approximately one-third of both the fine and coarse aggregate was added. At that point, all of the cement was ribbon-fed in with the remaining aggregate. Following that, all of the AEA, then the remaining water (less wash-down) and remaining HRWR were added. Typically 3 to 5 gal (10 to 20 L) of batch water, depending on the batch size, were held back for washing down the hopper during plant mixing. After mixing for a minimum of 70 revolutions at the plant, the mixture was checked, at least visually, and sent to the job site.

Trial batches, produced the week prior to placement, involved 2 to 3 yd^3 (1.5 to 2.5 m^3) of concrete used for non-structural working pads or storage slabs. The trial batches were used to refine the mixture for actual raw materials used and to allow the concrete supplier, the contractor's crew, and the local NCDOT staff to get a feel for what to expect from the mixtures. Desired strength levels were obtained.

These trial batches went reasonably well. However, variations in early truckloads during the first day of placement in the pavement indicated difficulty in mixing. Difficulty was experienced in discharging the last part of the load, and a lack of homogeneity throughout the truckload was noted. Therefore the subsequent batch size was reduced such that the maximum batch size was no more than half the rated mixing capacity of the truck. This change seemed to solve the mixing problems. In hindsight, the use of full-size trial batches would have been preferable.

The batch plant was close to the job site; travel time was usually about 5 minutes and rarely as much as 10 minutes. Once on the job site, the truck would turn around and back up close to the point of placement, next to a scaffold where the DCI had been premeasured and was waiting. The DCI was then added by hand with additional mixing, and the load was discharged by chute in front of a bridge deck machine. The concrete was then struckoff, screeded, and floated by the machine running on rails. Some hand-floating followed the machine, which was then followed by a burlap drag. The surface was tined as soon as possible, then immediately covered with curing compound.

If the section was to be insulated, plastic sheets were spread on the surface once it was tack-free;precautions were taken to prevent the sheets from wrinkling. A sheet of 1 in. (25 mm) thick rigid-foam building insulation was then placed on the slab, and boards, sandbags, or other weights were placed on the insulation to hold it down. Insulation was removed at the end of 6 hours.

No cracking due to plastic shrinkage was noted on any of the slabs. Although the high humidity certainly contributed to this, two other factors were important. First, the NCDOT field managers and the concrete crew were both anxious to get the surface finished and curing compound applied as soon as possible. Also, the contractor would mist the area immediately above the concrete from time to time. Since this was a nonbleeding mix, misting was felt to be particularly beneficial. Second, the mixtures were probably gaining strength faster than they were losing moisture, with shrinkage-induced stresses less than the strength of the concrete. However, premature cracking due to delayed sawing was a problem in two cases.

Sawing was typically done at the end of the work day, except in two cases. In the first case, the saw malfunctioned when the first section was ready to be cut, and sawing was delayed until the next day. In the second case, the placement lasted longer than expected, and sawing was delayed until the next day. In both cases, the slab had either cracked already or cracks ran out ahead of the saw blade. It is interesting to note that crack spacing was approximately 15 ft (4.5 m) on center. Delaying sawing of conventional concrete past 12 hours will frequently result in crack development. With high performance concrete this tendency may be increased due to volume changes of the concrete at very early ages as it undergoes significant temperature drops.

7.5.4.1 Construction Sequence

As noted above, the most extensive testing was conducted on concrete placed in the inside lane (see Figure 7.4). Three 60 ft (18.3 m) sections were placed on three separate days, for a total of 180 linear ft (54.9 m) of single-lane pavement. The 60 ft (18.3 m) length was chosen because this was the maximum length of rails available for the paving machine. Batch sizes were generally smaller than required to complete both placement and testing. Therefore, test panels would be placed as desired, and additional concrete of the same general type was used to complete the section for that day.

Four test panels were placed during the first day. Two panels were insulated and two remained uncovered. The naphthalene HRWR was used in all of the concrete placed on the first day. One of the insulated panels was placed using the CG mixture (HCI) and the other using the MM mixture (HMI). Similarly, one of the noninsulated panels was placed using the CG mixture (HCN) and the other using the MM mixture (HMN).

Following a rain delay, two test panels were placed on the third day in the next 60 ft (18.3 m) section. The concrete placed in this section contained CG only. These mixtures also used the melamine HRWR, as did all of the remaining high performance concrete in this installation. One panel was insulated (VCI) while the other panel was noninsulated (VCN).

The remaining test panels were placed on the fifth day, again after a rain delay, in the last 60 ft (18.3 m) section placed on the inside lane. The concrete in this section contained MM. Again, one panel was insulated (VMI) while the other panel was noninsulated (VMN).

The outside lane, as noted above, was placed in 1 day during the following week. Both CG and MM mixes were used. Only routine testing was conducted on these mixtures, since the primary purpose was to examine the effects of routine production rates on variability of the concrete as delivered.

7.5.4.2 Testing Plan

Slump and air content were determined for each batch. Air content was determined by the pressure method. For each test panel, 4 x 8-in. (100 x 200-mm) cylinders were fabricated for testing at 6 hours, 1 day, 7 days, 28 days, 6 months, and 1 year, using plastic molds with tightly fitting lids. Where the concrete was to be insulated, the cylinders were also initially insulated. Beams of 4 x 4 x 16 in. (100 x 100 x 400 mm) were cast in some cases. Specimen fabrication was generally accomplished by members of the research team, although NCDOT personnel assisted in some instances.

Cylinders were insulated by placing them in specially constructed insulating boxes described in Chapter 6 of volume 2 of this report series. Cylinders were transported to the NCDOT testing laboratory in Williamston, close to the bridge site, either just prior to testing at 6 hours for insulated cylinders or at about 20 hours for all other cylinders. All insulated cylinders were removed from the insulating boxes at 6 hours, regardless of age at testing. All cylinders were tested at the Williamston NCDOT laboratory at the appropriate age. Cylinders were stored outside the NCDOT laboratory until tested. The cylinders remained in the plastic molds after the lids had been removed at 1 day. The cylinders were thus exposed to conditions that were virtually identical to the exposure of the test panels at the bridge. Although at other field installations specimens were buried next to the highway, storage at the NCDOT laboratory was felt to be preferable since construction was still continuing and the laboratory was only a few miles away from the bridge.

A number of thermocouples were placed in the pavement to monitor curing temperatures and temperature gradients. Type T copper-constantan thermocouples were placed along the longitudinal centerline of the lane, 6 ft (1.8 m) from the edge, and were placed at either mid-depth of the slab or at mid-depth plus top and bottom quarter points, depending on location. Temperatures were taken with a hand-held digital thermometer.

Rapid chloride permeability tests (RCPTs) were performed in accordance with AASHTO T 277 (see Section 6.5.1) on specimens cut from the top of cores removed from the slab. The remaining portion of the core was trimmed to a length/diameter ratio as close to 2 as possible, then tested in compression.

Some flexural testing of beams was included. However, test results from the beams proved to be extremely variable and results should be viewed with great caution. In order to insulate these specimens, 1 in. (25 mm) thick rigid foam insulation was placed on the inside of standard 6 x 6 in. (150 x 150 mm) beam forms. While this caused little difficulty in the laboratory, consolidation of the concrete in the forms proved difficult at the project site. As noted earlier, 4 x 4-in. (100 x 100-mm) beams, insulated on the outside, have proven to be more successful.

7.5.5 Results and Discussions

The batch data for all mixtures are given in Tables 7.13 through 7.17. The data for the test panels are summarized in Tables 7.18 and 7.19. Cylinder and core strengths are the averages of two specimens. Duplicate cores were used to obtain specimens for RCPT rather than two slices from the same core. RCPT data are included in Table 7.19 with the other core data.

In addition to the data shown, cores from the test panels were to be tested at 28 days. However, due to a misunderstanding between the NCDOT engineer and the research staff, the 28-day cores were taken at the wrong locations, except for two panels.

A total of 17 batches of concrete were produced during the 3 days of placement in shorter sections at a slow construction rate, and a total of 16 batches were produced in the 1 day placement of the long section. Slumps during the 3 days of extensive testing ranged from 1¾ to 6 in. (45 to 150 mm), with an average of 2 3/4 in. (70 mm). Air contents of concrete during the same testing period ranged from 4.2% to 9.2% with an average of 6%. Standard deviations were 1 1/4 in. (30 mm) for slump and 1.4% for air content.

All of the concrete placed in the single-day 180 ft (54.9 m) section was to be noninsulated, and somewhat higher dosages of HRWR were used to increase the slump. Slumps of nonrejected loads during this phase of construction ranged from 2 to 9 1/2 in. (50 to 240 mm) with an average of 5 in. (130 mm). Air contents ranged from 4.4% to 10.3% with an average of 7%. Standard deviations were 2 in. (50 mm) for slump and 2.0% for air content, which were somewhat higher than during the series of slower placements.

Several truckloads of concrete were rejected during both the slow and rapid placements due to low slump and low air content. It is believed that changes in the moisture content of the aggregate, which were not easily detected since the plant did not have a moisture meter, were the primary cause of this problem. The situation may have been exacerbated by the batch sequence or by allowing the truck driver to wash down the loading chute after charging the drum, or both.

Table 7.18 Plastic properties of concrete, primary test sections, North Carolina

Test Section ID	HCI	HMI	HMN	HCN	VCI	VCN	VMI	VMN
Batch quantity (yd^3)	6	6	4	4	4	6	6	6
Slump (in.)	3.5	2.25	2.25	6	3	3.5	3	1.75
Air (%)	6.1	5.5	6.8	4.9	6.2	5.7	8	6.4
Cylinder temperature (°F)								
At placement	92	91	92	91	92	88	92	96
Maximum at	na	142	141	135	na	na	na	na
time (Hours)*	na	8.5	9	9	na	na	na	na
Slab temperature (°F)								
At placement	90	93	93	na	90	92	93	96
Maximum at	na	152	125	na	138	135	na	na
time (Hours)*	na	8.5	9	na	7.5	7	na	na

na = data not acquired, 1 in. = 25 mm, 1 yd^3 = 0.76 m^3; °C = (°F – 32) / 1.8;
*Maximum at time (Hours) means maximum temperature reached at xx hours after placement.

Table 7.19 Properties of hardened concrete, primary test sections, North Carolina

Test Section ID	HCI	HMI	HMN	HCN	VCI	VCN	VMI	VMN
Cylinder strength (psi)								
6 Hours	2,320	3,260	2,200	1,780	2,160	1,340	420	380
1 Da	5,280	6,090	4,850	4,570	5,170	5,290	4,820	4,700
7 Days	6,830	7,220	6,750	6,580	6,610	7,030	5,920	6,340
28 Days	7,600	7,680	7,310	7,760	7,000	7,360	6,100	6,570
6 Months	8,960	9,180	8,260	9,220	8,440	9,230	7,270	7,610
12 Months	9,470	9,320	8,010	9,550	8,900	9,220	7,150	7,670
18 Months	10,210	9,020	8,580	10,250	9,510	9,750	7,400	7,960
Flexural strength (psi)								
6 Hours	na	380	450	na	455	195	165	75
1 Day	na	515	530	na	630	610	495	505
7 Days	na	570	640	na	780	na	520	505
Compressive strength of core (psi)*								
6 Months	8,040	7,670	7,180	9,080	8,520	8,700	6,640	7,660
12 Months	8,750	7,470	6,890	9,700	8,910	9,080	6,620	8,030
Rapid chloride permeability test on core (coulombs)								
6 Months	3,430	2,850	2,190	2,780	1,810	2,870	2,320	2,660
12 Months	1,650	3,160	1,150	2,000	1,510	1,030	2,260	1,810

*Compressive strength from 4-in. (100-mm) diameter cores adjusted for l/d, 1,000 psi = 6.9 MPa,
Rapid chloride permeability test results were adjusted for area.

Except as noted above, air contents and slumps were generally within desired limits. However, variability in slump and air content during the long placement was higher than that during the shorter, slower placements. Although this is probably related to some reduction in the level of batch-to-batch control due to higher production rates, it is very likely that a large part of this variability, along with generally higher values of slump and air content, was simply due to the variability in the amount of time during which the trucks were held at the plant for testing and adjustment. The effects of time on slump loss, with a consequent loss in air content, when using HRWR is consistent with prior observations.

Strengths of insulated panels met the 2,000 psi (14 MPa) criterion for VES mixtures at 6 hours in all but one case. Strengths of noninsulated panels were also generally good and, in one case, also met the 6-hour strength criterion. This is almost certainly due to the high ambient temperatures at the time of concrete placement. Two mixtures, VMI and VMN, did not meet the required strength at 6 hours. These batches contained a very large quantity of HRWR, which caused retardation of the concrete and low early strengths, although strengths at 1 day were comparable to the other mixtures.

Strengths at 1 day did not all meet the criterion for 24-hour strength but were fairly close. The average 1 day cylinder strength for the test panels was greater than 5,000 psi (34 MPa).

Average 28-day cylinder strength was about 7,300 psi (50 MPa), and average 6-month cylinder strength was about 8,600 psi (59 MPa). Cylinder strengths at 1 day averaged 71% of 28-day strength, strengths at 7 days averaged 93% of 28-day strength, and strengths at 6 months averaged 118% of 28-day strength. Twenty-eight-day strengths correlated best with air content alone.

After 28 days, and particularly after 6 months, the data were best analyzed as two groups based on coarse aggregate type. At later ages, CG mixtures had higher strengths than did mixtures produced with MM. The CG concretes continued to gain strength at all ages. Strengths of the MM mixtures reached the upper limit of potential strength at about 6 months. The apparent upper limit of just over 9,000 psi (62.1 MPa) exhibited by the MM mixes is consistent with the coarse structure and the relatively high porosity of the aggregate.

As expected, at later ages similar mixtures with the naphthalene-based HRWR tended to have higher strengths than those with the melamine-based HRWR.

Both cylinder and core data indicate continued strength gain with time, under typical field conditions and without exposure to continuous moist curing. Core strengths at 6 months virtually matched cylinder strengths at 28 days. The ratio of core strength at 12 months to cylinder strength at 28 days was approximately 114%.

Although, as mentioned earlier, cores were not available for all panels at 28 days, two panels were cored. At 28 days, a single core of the HCI panel had a strength of 7,680 psi (53 MPa), compared with 7,600 psi (52 MPa) cylinder strength. The VCN panel had 28-day strengths of 7,980 psi (55 MPa) for two cores and 7,180 psi (50 MPa) for cylinders.

Core strengths at both 6 and 12 months compared very favorably to cylinder strengths, except in two panels. Ratios of core strength to cylinder strength ranged from 90% to 101% at 6 months and 92% to 105% at 12 months, except for the HMI and HMN panels. For the concrete in these panels, the ratios of core strength to cylinder strength were in the low to mid-80s at both ages. Recall, however, that according to ACI 318 (1993b), concrete in structural members is considered to be acceptable if individual core strengths are at least 75% and average core strengths are at least 85% of required cylinder strength.

RCPT results indicate the concrete is moderately permeable (2,000 to 4,000 C; see Table 7.19) to the ingress of chlorides at 6 months, with results generally on the low side of the limits. Although most of the samples indicated a reduction in chloride permeability between 6 and 12 months, samples from one test panel were slightly more permeable than at 6 months. No visible indications of problems were detected in this section of pavement at 18 months, however.

The RCPT data at 12 months indicate the concrete has a low (1,000 to 2,000 C) to moderate (2,000 to 4,000 C) chloride permeability. At both 6 and 12 months, there was no clear pattern to RCPT values based on insulated vs. noninsulated panels, nor based on aggregate or HRWR type; nor were the RCPT data found to be correlated with strength or with strength gain.

The differences in RCPT results between 6 and 12 months are highly variable and may be indicative of inherent variability in the test method, the necessarily broad interpretation of test results, differences in salt content of the pavement due to leaching of calcium nitrite or deposition of salt, variability of the concrete as placed, or other factors. Note that the test variability for duplicate cores was less than 100 C, indicating consistent testing.

The RCPT results should be viewed with caution, since the presence of any soluble salt (such as calcium nitrite) will increase the measured value by reducing the resistivity of pore solution in the concrete. The concrete is therefore probably less permeable than RCPT data would indicate.

7.5.6 Follow-up Site Visit

During the follow-up visit to the Williamston site, no particular difference was noted in surface quality based on visual inspection between slabs placed using conventional concrete and those placed using high performance concrete. Except for cracks that had occurred within 24 hours of placement due to delay in sawing joints, no additional cracking of the pavement was detected. Those cracks had been sealed almost immediately and appeared to be holding up well even though some were fairly wide. Slight raveling of one sawed joint was detected, but it appeared to be minor.

7.6 Test Pavement — Illinois

7.6.1 Site Description

The third field installation is on Interstate 57 about 10 miles (16.1 km) north of Effingham, Illinois. The site is situated on the line between Shelby and Cumberland Counties at mile marker 172. Two patches were placed on October 24, 1991 in the outside lane of the northbound traffic. A distance of about 1,000 ft (305 m) separates the two patch sites.

I-57 is a four-lane divided highway crossing very flat terrain in rural Illinois. The roadway was constructed and opened to traffic in 1964. The traffic lanes are 12 ft (3.7 m) wide with 10 ft (3.1 m) outer and 4 ft (1.2 m) inner shoulders. The original pavement is approximately 10 in. (250 mm) of portland cement concrete over 6 in. (150 mm) of granular base. Welded wire fabric was used in the original construction. Contraction joints are spaced every 100 ft (31 m). Epoxy-coated dowels are used for load transfer. The ADT for this roadway is roughly 11,800 with approximately 22% trucks.

The Illinois installation was part of a large repair/maintenance contract. Repairs were being made primarily on I-70, which also passes through Effingham. However, in the area of the installation the pavement appeared to be in fairly good condition.

7.6.2 Patch Description and Preparation

This installation consists of two patches that resemble the "typical" patch described in Section 7.3. The patches are 45 ft (13.7 m) long, 12 ft (3.7 m) wide, and about 10 in. (250 mm) thick.

The existing concrete was prepared for removal on October 23, 1991. A large-diameter saw and tractor-mounted drill were used in this operation. The concrete was removed using chains and a loader. Epoxy-coated dowel bars were placed at all transverse joints. There were no caps on the dowel bars. Dowel bars were grouted into the existing concrete. Both patches were constructed with welded wire fabric.

Thermocouples were placed in each section of both patches. A channel was cut into the asphalt shoulders so that the thermocouple wire could be run out away from the edge of the pavement. These channels were later filled in with an asphalt mixture to protect the wires. At the time the concrete was placed all thermocouples functioned properly. However, the labels used to identify the wires as top, mid-depth, or bottom were discovered missing when 28-day testing was conducted.

In addition to the two experimental patches, a small full-lane-width and full-depth patch was constructed for trial batching. This patch was constructed to repair a failed joint. Welded wire mesh was not used, but grouted and epoxy-coated dowel bars were installed at both faces of the existing concrete. On removal of the old concrete it was discovered that the dowel bars had lost a significant proportion of their cross-section.

7.6.3 Materials and Proportions

The HES mixture used in Illinois and all subsequent installations was an updated version of the mix used in the New York installation. The proportions provided to the state are those given in Table 7.1. Table B.4 gives the properties of aggregates, and Table B.5 shows the proportions used by the Illinois Department of Transportation (ILDOT) in its laboratory trial batching of the HES mix. Their coarse aggregate was a limestone, and all admixtures were W. R. Grace products. Table B.6 gives the fresh and hardened concrete properties for the trial batch. Both insulated and noninsulated cylinders were tested. The strength data presented are the averages of two cylinders. At 5 hours the insulated cylinders were more than 30% stronger than those not insulated. The insulated cylinders also exceeded the minimum strength requirement of the VES mix at 6 hours. After 24 hours the insulated cylinders were only about 10% stronger than those not insulated.

A small field trial batch was produced on October 23, 1991 so that adjustments could be made to the admixture dosages prior to construction of the installations. All material quantities were consistent with those in Table 7.1. The only deviation was the dosage rates for the HRWR and AEA. The dosage rates for these admixtures were 18 oz/cwt (11.75 mL/kg) for the HRWR and 5 oz/cwt (3.26 mL/kg) for the AEA. Based on the fresh concrete properties of the trial section, the dosage rate for the AEA was decreased to 3.5 oz/cwt (2.28 mL/kg). Thus, for all six truckloads of concrete produced, the dosages of the admixtures were 18 oz/cwt (11.75 mL/kg) for the HRWR and 3.5 oz/cwt (2.28 mL/kg) for the AEA.

7.6.4 Batching, Placing, and Curing

Based on the size of the patches and the number of test specimens to be cast, trucks were charged with 7 yd^3 (5.4 m^3) of concrete. The final truck contained only 4.5 yd^3 (3.4 m^3) of concrete, as this was all that was needed to complete construction of the second patch.

Unlike any of the other installations, a central mixer was used in Illinois. Batching was done from a control room across the yard. The Type III cement was stored in a silo and was batched using normal procedures. Two loads, each about 3.5 yd^3 (2.7 m^3), were batched for one truckload of concrete. This was due to limited scale capacity. The elapsed time between loads was about 5 minutes. The air content of the first load of concrete was checked at the batch plant. Based on the results, adjustments were made to the dosage of AEA for the second load. The two loads were mixed and the truck left the yard for the job site. The proportions for all six truckloads of concrete are given in Table 7.20.

Travel time from the batch plant to the job site was approximately 30 minutes. This involved both city and highway travel. When the truck arrived at the job site, the calcium nitrite solution was added manually with 5 gal (18.9 L) buckets. The concrete was then mixed and discharged directly into the patch.

Table 7.20 Actual batch weights and fresh concrete properties — Illinois

Material	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	Truck 6
Btach Size (yd^3)	7.0	7.0	7.0	7.0	7.0	4.5
Cement (Type III) (lb)	6,065	6,055	6,070	6,055	6,070	3,910
Water (lb)	1,375	1,383	1,283	1,333	1,383	883
Coarse Aggregate (lb)	12,200	12,160	12,240	12,220	12,130	7,880
Fine Aggregate (lb)	6,590	6,700	6,580	6,580	6,500	4,030
HRWR (WRDA-19) (oz)	1,100	1,100	1,100	1,100	1,100	780
AEA (Daravair) (oz)	210	210	210	204	204	130
Calcium Nitrite (DCI) (gal)	28	28	28	28	28	18
Slump (in.)	4.0	3.25	3.0	2.5	3.5	2.75
Air Content (%)	8.7	10.5	8.7	7.8	10.2	7.2
Unit Weight (lb/ft^3)	138.1	137.1	139.2	141.7	137.3	142.1

Construction of this installation began with the insulated section. This patch is the southernmost of the two. The concrete from Truck 1 was placed in the southernmost section of the insulated patch. Work proceeded for both patches in the direction of traffic. Concrete from Trucks 1 through 3 was used for the insulated patch; concrete from Trucks 4 through 6 was used for the noninsulated patch.

The concrete was consolidated using a spud vibrator. A mechanical vibrating screed was also used. A broom finish was applied to the surface. The insulated patch was covered with blankets having an R12 insulation rating. The insulation remained on the patch approximately 4 hours. Saw cuts were made at the joints with a hand-held saw after the concrete had set sufficiently. The patches remained closed to traffic overnight.

One problem in Illinois was a lack of information regarding the moisture content of the aggregate. Although the total moisture content was determined at the batch plant, the absorption capacity of the rock and sand were not known, hence the actual amount of water contributed to the mix by the aggregate was not known. Adjustments were made for the aggregate moisture by holding back some water at the batch plant, making a visual estimate of the slump, and adding a few gallons to the truck for low slumps.

7.6.5 Results and Discussions

Properties of the fresh concrete were determined by ILDOT personnel. Cylinder and beam specimens were cast by both University of Arkansas and ILDOT personnel. Two specimens were cast for each test age.

All specimens were cured on site in polystyrene curing boxes. Two 4 x 8-in. (100 x 200-mm) and two 6 x 12in. (150 x 300-mm) cylinders, cast from the concrete from the second truck for each patch (Trucks 2 and 5), were cured in air for 24 hours and then brought to an ILDOT office for standard curing until tested. Cylinders cast for early-age testing were transported to the ILDOT office in Effingham by the research staff. These cylinders were not moved until they were about 3 hours old.

The slump and air content of the trial batch were 7 in. (175 mm) and 10%, respectively. These values were higher than desired. As a result, the dosage rate for the AEA was reduced from 5 to 3.5 oz/cwt (3.26 to 2.28 mL/kg). It was felt that decreasing the dosage of the AEA would reduce the slump and air contents to more acceptable levels. The dosage rate of the HRWR was not altered.

7.6.5.1 Fresh Concrete Properties

The fresh properties for all truckloads of concrete are given in Table 7.20. Slumps ranged from 2.5 to 4.0 in. (63 to 100 mm), averaging about 3 in. (75 mm). The slumps were within the target range of 3 to 5 in. (75 to 125 mm). The air contents ranged from 7.2% to 10.5%, averaging about 9%. Even with the reduction in AEA from 5 to 3.5 oz/cwt (3.26 to 2.28 mL/kg), the air contents were on the high side of the 5% to 8% target range. Minute adjustment of the admixture dosages in the field is not practical, so further changes in the dosage rates were not made. The unit weights were low, which is consistent with the higher air contents.

The first truckload of concrete brought to the site for the second patch (Truck 4) had a very high air content, about 15%. This was not detected until about 1 yd^3 (0.8 m^3) of material had been placed in the patch. The truck was rejected, but the concrete was not removed from the patch. The cause of the high air content was a malfunction in the admixture dispensing equipment at the batch plant. The reason the last load of concrete was only 4.5 yd^3 (3.4 m^3) was, in part, a result of the material left in the patch from Truck 4.

Figure 7.5 shows that the temperature histories for the insulated patch and corresponding cylinder are very similar. This would suggest that the insulated cylinders experienced curing conditions similar to those of the insulated patch. Figure 7.6 presents the temperature data for the noninsulated patch and cylinder. The temperature history for the cylinder is more in line with that of the top of the patch rather than the mid-depth. Because the noninsulated patch was placed later in the day than the insulated patch, the ambient temperature conditions may have led to heat loss at the surface of the patch. Due to the time at which work was completed, temperature data are available only up to 3 hours for the noninsulated patch. As a result, it is difficult to make any firm comments on the meaning of Figure 7.6.

7.6.5.2 Hardened Concrete Properties

Tables 7.21 through 7.24 present data from early-strength cylinder testing, long-term cylinder testing, and coring, respectively. The apparatus used in Illinois for testing beams is designed for

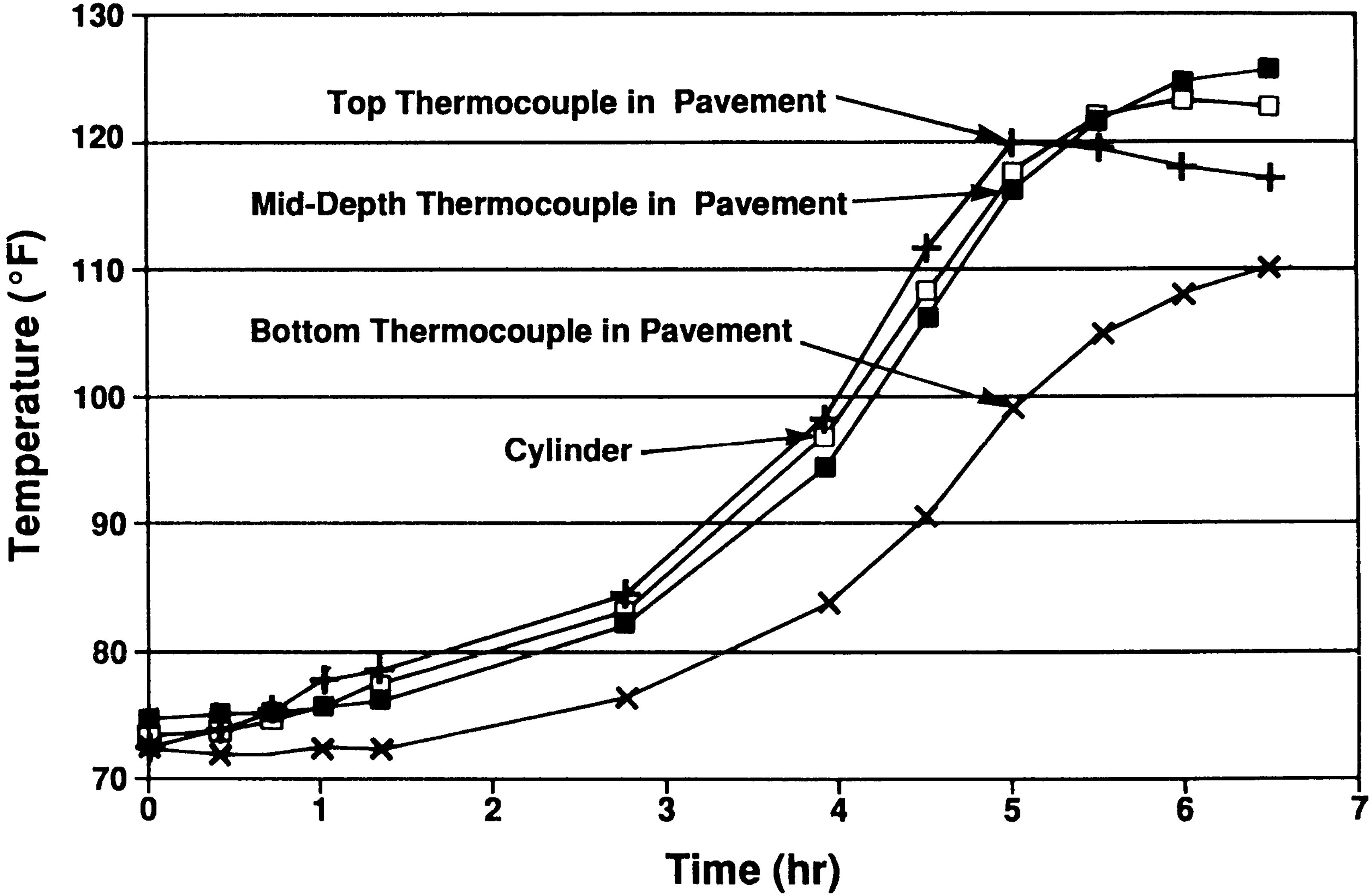

Figure 7.5 Truck 2 concrete temperature data (insulated) — Illinois

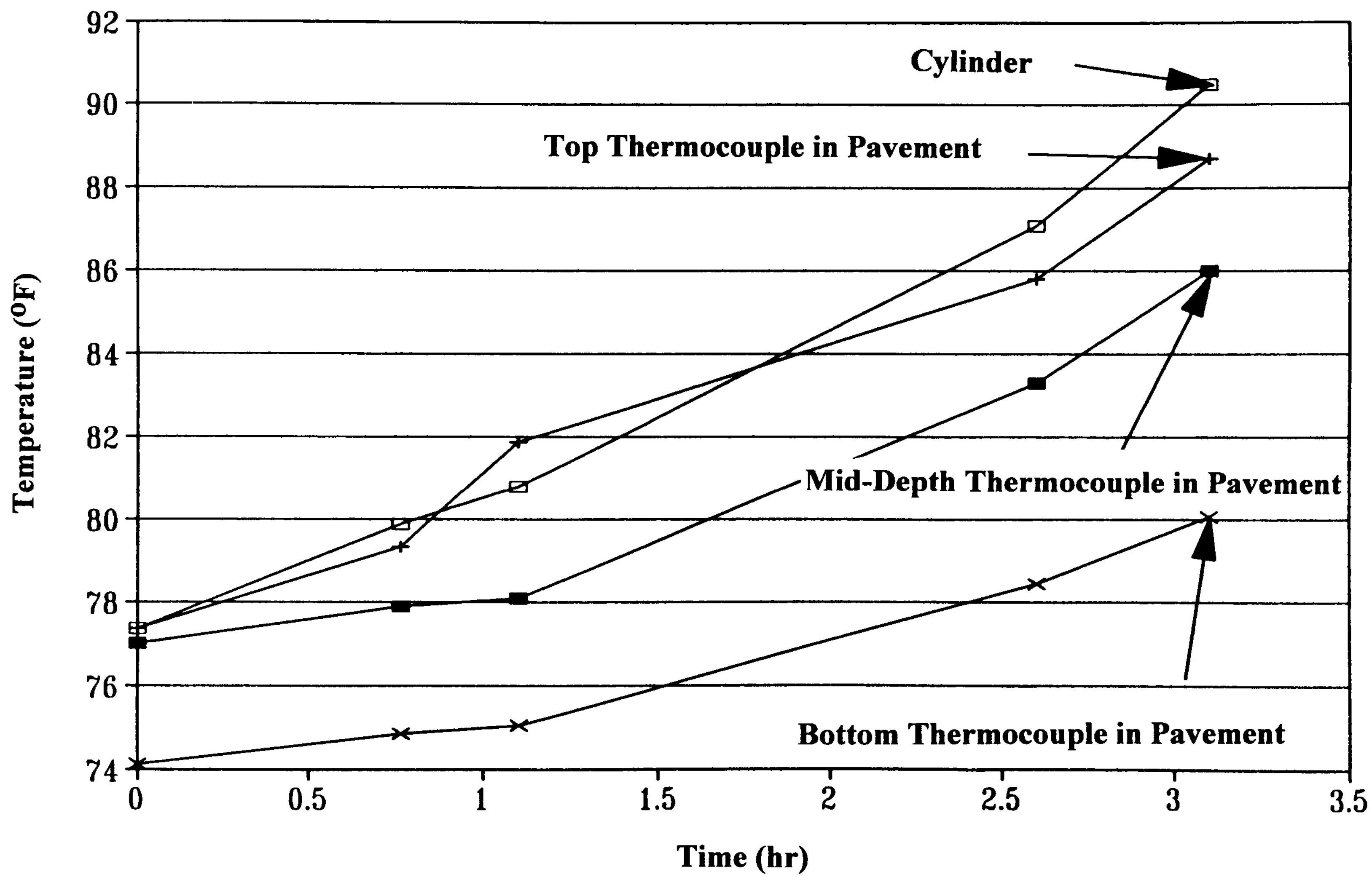

Figure 7.6 Truck 5 concrete temperature data (noninsulated) — Illinois

Table 7.21 Early strength of 4 x 8-in. cylinders (psi) — Illinois

Test Date	Test Age	Truck 1	Truck 2	Truck 3
10/24/91	4 Hours		40 80	
	5 Hours		550 800	
	6 Hours	1,350 1,430	1,510 1,530	1,750 2,050
	7 Hours		2,010 2,470	

Table 7.22 Long-term strength of 4 x 8-in. cylinders (psi) — Illinois

Test Date	Tests Age	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	Truck 6
10/25/91	1 Day		5,010 4,770		5,730 4,890	5,010 —	5,570 6,050
10/31/91	7 Days	6,410 5,690	6,680 6,130	6,760 5,890	6,760 6,840	6,610 6,490	7,680 7,360
11/21/91	28 Days	6,370 6,570	5,650 6,610	6,880 6,960	6250 7,000	7,000 6,130	— 6,680
4/24/92	6 Months		4,380 5,050			5,090 5,970	
10/30/92	12 Months		9,500 9,920			9,850 8,680	
5/3/93	18 Months		9,870 9,270			8,400 9,290 9,080	

Table 7.23 28-Day strength of laboratory-cured cylinders (psi) — Illinois

Cylinder Size	Truck 2	Truck 5
4 x 8 in.	6,490 7,000	8,360 8,280
6 x 12 in.	7,550 6,880	7,990 7,500

Table 7.24 Strength of 4-in. cores (psi) — Illinois

Test Date	Test Age	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	Truck 6
11/22/91	29 Days	7,170	6,430 5,880 6,590	6,340	7,990	5,280 5,650 4,890	7,790
4/24/92	6 Months	7,890	7,750 7,380 7,780	7,720	6,940	7,340 8,470 7,110	7,960
10/30/92	12 Months	8,640	7,010 8,300 7,860	7,510 7,610	10,070	6,340 5,820 6,780	8,520
4/93	18 Months	7,660	8,490 8,720 6,680	7,410	10,690	7,750 7,840 7,220	10,380

a center loading on 6 x 6 x 18-in. (150 x 150 x 450-mm) beams. Attempts were made to modify the apparatus for third-point loading on 4 x 4 x 12-in. (100 x 100 x 400-mm) beams, but none were satisfactory. As a result, no data are reported for the beams cast in Illinois because they could not be properly tested.

A problem was encountered in Illinois with the compression testing of 4 x 8-in. (100 x 200-mm) cylinders. Neoprene pad caps were used to test these cylinders. In testing, a number of cylinders failed prematurely at their ends when pieces of concrete broke away from the edges of the cylinders. The resulting measured strength was very low relative to companion cylinders that did not fail in this way. As a result, some test data were discarded. Dashes are placed in the tabulated data for cylinders that were known to have failed in this manner.

Early-age testing of concrete from the insulated patch indicates that this material did not meet the requirement of 2,000 psi (13.8 Mpa) in 6 hours for the VES mix (see Table 7.21). Strengths for the three truckloads of concrete averaged 1,390 psi (9.6 MPa), 1,570 psi (10.8 MPa), and 1,900 psi (13.1 MPa) at 6 hours. The trend in strengths, low for Truck 4 and higher for Truck 6, can be explained in part by the fact that the patch was not insulated until the third section was complete. Thus, when the concrete from Truck 4 was 6 hours old, it had only been insulated about 4 hours. The same is true for the concrete from Truck 5, which was insulated only 5 hours when tested at 6 hours. Only the concrete from Truck 6 had been insulated for the full 6 hours when it was tested at 6 hours. Data for Truck 5 indicate that this material gains strength at a rapid, but fairly steady, pace. Note that this concrete was placed in late October when ambient temperatures were low. Thus, the low strengths within the first few hours can be partly attributed to heat loss to the environment.

Strength data at 1 day for both patches are in line with the requirement for the HES mix of 5,000 psi (34.5 MPa) in 24 hours (see Table 7.22). On average, the measured strength for the noninsulated patch was higher than that for the insulated patch. This is unexpected but can be explained by the air contents of the concrete. Trucks 2 and 5, which had similar high air contents, also had similar low 1-day strengths. Trucks 4 and 6, which had similar low air contents, had similar relatively higher 1-day strengths. Thus, if 1-day test cylinders were cast from Trucks 1 and 3 (insulated), their strengths may have been higher than those of Trucks 4 and 6 (noninsulated).

The measured strength data are erratic from 28 days through 12 months. Data collected at 28 days and at 6 months indicate a drop in the measured strength. It is believed that these low strength values are due to the failure of the cylinders at their corners, as discussed above. After the 6-month testing, ILDOT personnel were instructed to continue loading the cylinders until they completely failed. The 12-month data, which show a significant increase in measured strength, would suggest that the testing method, not the concrete, produced the low 28-day and 6-month strengths.

Results for cylinders that were cured in the laboratory appear to support the conclusion that the test method, not the concrete, caused the low strengths. Table 7.23 presents 28-day data for both 4-in. (100-mm) and 6-in. (150-mm) cylinders. Although strength variations among cylinders are high, the magnitudes of the strength values are in line with what would have been expected for the field-cured cylinders at 28 days or 6 months.

Core data are also erratic, especially for the noninsulated batch (see Table 7.24). Ignoring the 6-month cylinder data, the measured strength of the cores is well below that of the cylinders for the noninsulated patch. For the insulated patch, the cylinders and cores had relatively similar strengths at 28 days. The difference between them has increased over time, with the cylinders being much stronger.

7.6.6 Follow-up Site Visit

The Illinois installation was visited in late January 1993. Three very fine transverse cracks were discovered during the inspection (see Figure 7.7). One crack was in the center section of both patches. The third crack was in the northern section of the insulated patch. All three cracks intersected a core hole. The crack in the center section of the insulated patch extended from both sides of the core hole. Only the crack in the center section of the noninsulated patch extended out to the edge of the patch. In addition to these cracks, there appeared to be evidence of shrinkage cracking over isolated areas of both patches.

The surface of the hardened concrete showed signs of the concrete being sticky when placed. Aggregates were protruding from the surface. Some had a trail from being dragged across the surface.

All joints appeared to be in good condition, as did the remainder of both patches.

7.7 Test Pavement — Arkansas

7.7.1 Site Description

Two weeks after the Illinois installation was completed, the next field site was constructed in Arkansas. The Arkansas installation is on I-40 less than 5 miles (8.1 km) west of Forrest City. There are two patches in the passing lane of the westbound traffic near mile marker 237. The patches were placed on November 6, 1991. A distance of about 500 ft (152.4 m) separates the two patch sites. Signs provided by the Arkansas Highway and Transportation Department (AHTD) identify the experimental sections as HES and VES concrete.

This section of I-40 was opened to traffic in 1966. The roadway consists of two 12 ft (3.7 m) traffic lanes. Shoulders are asphalt and are 10 and 4 ft (3 and 1.2 m) for the outer and inner shoulders, respectively. The pavement is 10 in. (250 mm) of plain portland cement concrete over a cement-treated subgrade. Joints are spaced at 15 ft (4.6 m). Contraction joints are located at a 45 ft (13.7 m) spacing. The pavement is generally in good condition. There is minor faulting, 1/4 to 1/2 in. (6 to 13 mm), in the area of the installation.

The Arkansas installation was a repair project. Preparation of the site and construction of the patch were performed by the local district maintenance crew.

The climatic exposure in the area of the pavement can be described as wet with potential for freezing-thawing cycling. In 1989, the two-way AADT near the installation was determined to be 19,890. Trucks account for just over 47% of this traffic.

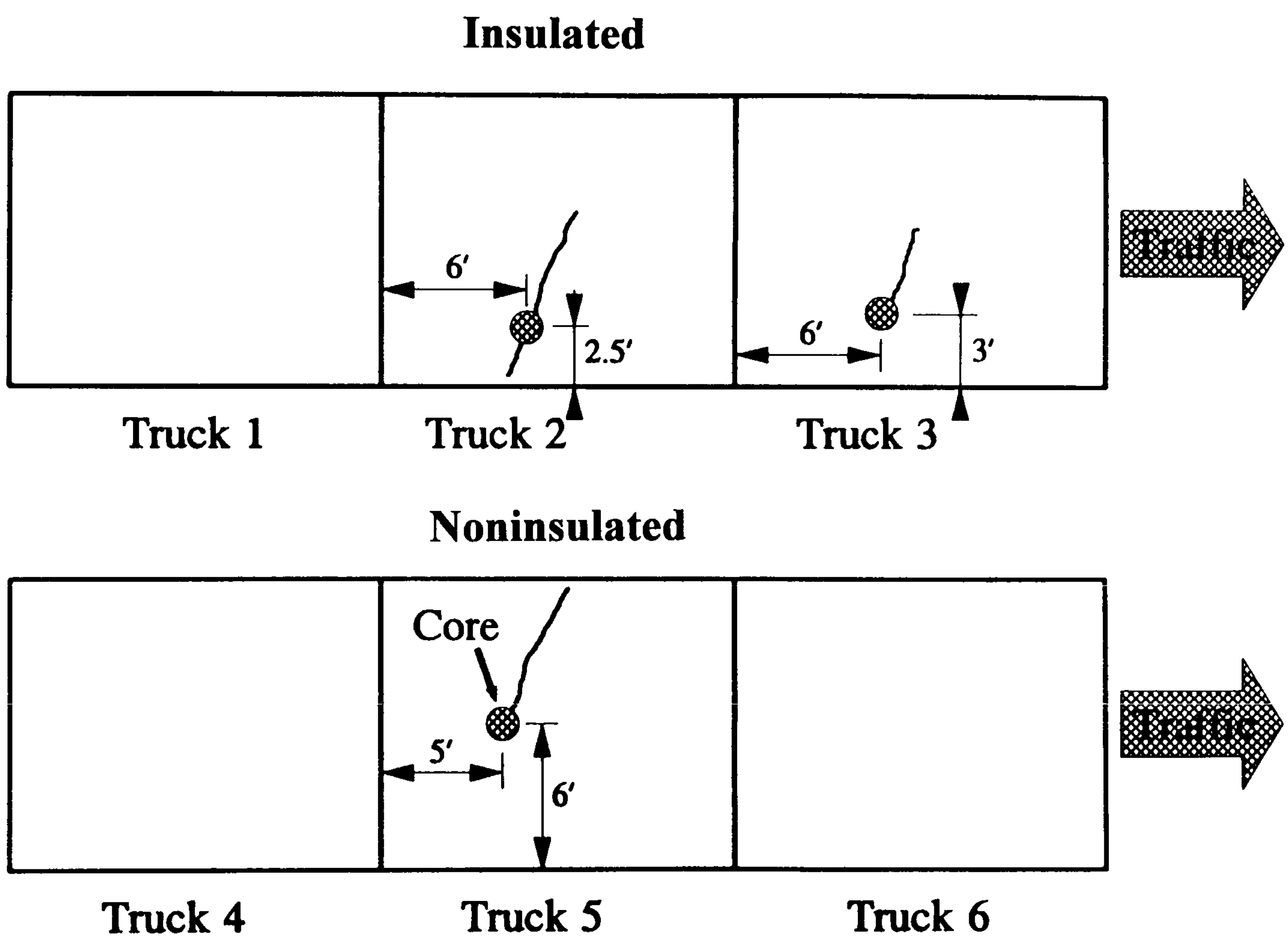

Figure 7.7 **Cracks in Illinois installation**

7.7.2 *Patch Description and Preparation*

The Arkansas installation consists of two patches 45 ft (13.7 m) long, 12 ft (3.7 m) wide, and 10 in. (250 mm) thick. These patches can be considered "typical" sections. See Section 7.3 for a general description of the site.

Removal of concrete from the repair area took place over a 1-week period. This occurred for two reasons. First, cold and rainy weather caused delays. Second, the coarse aggregate commonly used in the eastern part of Arkansas is quite hard. As a result, the life expectancy of saws and saw blades is rather short. During preparation of the area, one saw was damaged and a saw blade had to be replaced. Once cuts were made in the old concrete, its removal and preparation of the area proceeded smoothly.

The two patches were constructed with plain concrete. Thermocouples were placed in each section of both patches. Since the subbase at the patch sites was hard and no wire mesh was used, it was difficult to secure the thermocouples in position. Small flags were attached to the PVC pipe used to install the thermocouples so that they would not be disturbed during construction.

In addition to the two experimental patches, a small full-lane-width, full-depth patch, 6 ft (1.8 m) long, was constructed for trial batching. Welded wire fabric was used for this patch.

7.7.3 *Materials and Proportions*

The standard HES mixture (see Table 7.1) was used for the Arkansas installation. Since the proportions in Table 7.1 are based on laboratory tests conducted at the University of Arkansas, it was decided to use the same materials for this field installation. As a result, all constituent materials were trucked in for this work. Data on the cement, coarse aggregate, and fine aggregate used in Arkansas are given in Tables 7.25 and 7.26.

Proportions used to produce the small trial section in Arkansas are given in Table 7.27. The dosage rates for the HRWR were 18 oz/cwt (11.75 mL/kg) and 4.5 oz/cwt (2.94 mL/kg) for the AEA. Based on the air content of this concrete, the dosage for the AEA was increased to 5 oz/cwt (3.26 mL/kg) for the actual installation.

The slump for the first truckload of concrete for the actual installation was 6 inches (150 mm). This was higher than the target range of 3 to 5 in. (75 to 125 mm). Therefore, the dosage rate of the HRWR was reduced from 18 to 16 oz/cwt (11.75 to 10.44 mL/kg) for all subsequent trucks. All other materials, except the AEA as mentioned above, were batched according to Table 7.1.

Table 7.25 Type III cement properties -- Arkansas

Chemical Analysis	Percent
Loss on ignition	1.4
Silicon dioxide (SiO_2)	20.5
Calcium oxide (CaO)	65.2
Aluminum oxide (Al_2O_3)	6
Iron oxide (Fe_2O_3)	2
Magnesium oxide (MgO)	1.1
Sulfur trioxide (SO_3)	3.4
Total alkalis as Na_2O equivalent	0.38
Insoluble residue	0.5
Tricalcium silicate (C_3S)	58
Dicalcium silicate (C_2S)	15
Tricalcium aluminate (C_3A)	13
Tetracalcium aluminoferrite (C_4AF)	6

Table 7.26 Aggregate gradation and material properties — Arkansas

	Percent Passing	
Sieve Size	Coarse	Fine
1 in.	100.0	
3/4 in.	99.0	
1/2 in.	82.0	
3/8 in.	48.0	
No. 4	7.0	96.0
No. 8	4.0	89.0
No. 16	3.0	75.0
No. 30		54.0
No. 50		12.0
No. 100		0.5
Specific gravity	2.66	2.62
Absorption (%)	0.80	0.65

1 in. = 25 mm

Table 7.27 **Batch quantity for test patch — Arkansas**

Material	Total Quantity Per Batch
Batch size (yd^3)	3.0
Cement (Type III) (lb)	2,630
Water (lb)	710
Coarse aggregate (lb)	5,080
Fine aggregate (lb)	3,240
HRWR (WRDA-19) (oz)	470
AEA (Daravair) (oz)	117
Calcium nitrite (DCI) (gal)	12
Slump (in.)	5.0
Air content (%)	3.0
Stength at 5.5 Hours (noninsulated) (psi)	4,800

7.7.4 Batching, Placing, and Curing

For this installation, two of the three truckloads of concrete used for one patch were charged with 6 yd^3 (4.6 m^3) of concrete. The third truckload of concrete for each patch was to have 7 yd^3 yards (5.4 m^3) to compensate for material lost in sampling. When the final truckload was to be batched, it was discovered that there would not be enough Type III cement remaining for 7 yd^3 (5.4 m^3) of concrete. Therefore, two 3.5 yd^3 (2.7 m^3) batches were produced. The first load used only Type III cement. The second used 80% Type III and 20% Type I cement.

Dry batching of the concrete was done at a ready-mix plant in Forrest City. Travel time from the batch plant to the job site was approximately 15 minutes. This involved both city and highway travel. The batch plant did not have a silo available to store and dispense the Type III cement used in the HES mixture. As a result, the cement was batched directly from a tanker truck to the hopper. This turned out to be a major problem. The hose used to discharge the cement into the hopper had to be secured to the tower supporting the cement silo. The hose had to make a number of sharp turns to be properly positioned at the exit nozzle, thus creating kinks along its length. This seriously slowed the rate at which cement could be discharged. If no facility is available for storage and dispensing of Type III cement, it may be better to use cement in bags (as was done in New York) and to use a tanker (as in Arkansas) unless a system can be devised and checked prior to large-scale production.

Due to the limited scale capacity at the batch plant, it was necessary to split the weights of all materials in order to produce the concrete. With the effort required to batch the cement, it took a very long time to produce 6 yd^3 (4.6 m^3) of concrete. The first truckload took about 45 minutes to batch. With experience this time was quickly reduced to an average of about 30 minutes. Even so, a 30-minute batching time was too long.

As mentioned above, there was not enough Type III cement available to produce the roughly 42 yd^3 (32.1 m^3) required for the two patches and the field test patch. However, the ready-mix producer indicated that he had purchased enough cement for 50 yd^3 (38.2 m^3) of the HES mixture. Considering that 870 lbs (395 kg) of cement are used for each cubic yard, this means that nearly 7,000 lbs (3,175 kg) of Type III cement were unaccounted for. It is not clear whether the ready-mix producer did not receive the cement he ordered or whether the cement went into the concrete. The ready-mix producer felt that the material was lost as dust as the cement was blown into the hopper. Table 7.28 provides the batch weights for all seven trucks.

Construction of this installation began with the insulated section. This patch is the easternmost of the two. The concrete from Truck 1 was placed in the easternmost section of the insulated patch. Work proceeded for both patches in the direction of traffic. Thus, Trucks 1 through 3 were used for the insulated patch while Trucks 4 through 7 were used for the noninsulated patch.

When the truck arrived at the job site, the calcium nitrite solution was added manually with 5 gal (18.9 L) buckets. The concrete was then mixed and discharged directly into the patch. The concrete was consolidated using a spud vibrator. Burlap was dragged across the length of the patch. The surface was tined.

The insulated patch was covered with rigid foam insulation. The insulation was placed on the pavement around 1 p.m. and remained on the patch until approximately 4:30 p.m. The insulation was not placed on the patch until the concrete in the last section was about 3 hours old. At that time the first section of this patch was 4.5 hours old and the middle section 3.5 hours old. Joints were sawed in the insulated patch after the insulation was removed. Joints in the noninsulated patch were not sawed until the following morning. The patches were opened to traffic on the morning of November 9.

7.7.5 Results and Discussions

Properties of the fresh concrete were determined by AHTD personnel. Cylinder and beam specimens were cast by University of Arkansas personnel. Two specimens were cast for each test age. A few extra cylinder specimens were cast from the second and fifth truckloads of concrete. This allowed for three specimens to be tested at 7 days.

All specimens were cured on site in polystyrene curing boxes. Two 4 x 8-in. (100 x 200-mm) and two 6 x 12-in. (150 x 300-mm) cylinders, cast from the concrete from the second truck for each patch (Trucks 2 and 5), were cured in air for 24 hours and then brought to the AHTD Little Rock office for standard curing until tested. Cylinders cast for early-age testing were transported to an AHTD district office near Wynne, Arkansas, by the research staff. These cylinders were not moved until they were about 3 hours old.

Table 7.28 Actual batch weights and fresh concrete properties — Arkansas

Material	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	Truck 6	Truck 7
Batch Size (yd^3)	6.0	6.0	7.0	6.0	6.0	3.5	3.5
Cement (Type III) (lb)	5,215	5,205	6,065	5,285	5,220	3,050	*3,050
Water (lb)	1,425	1,425	1,675	1,417	1,417	833	833
Coarse aggregate (lb)	10,320	10,220	11,920	10,220	10,240	5,930	5,880
Fine aggregate (lb)	6,360	6,360	7,420	6,360	6,360	3,760	3,780
HRWR (WRDA-19) (oz)	956	818	975	835	840	488	495
AEA (Daravair) (oz)	260	260	305	263	260	153	158
Calcium nitrite (DCI) (gal)	24	24	28	24	24	15	15
Slump (in.)	6.0	2.5	6.0	3.5	4.5	4.0	6.0
Air Content (%)	6.0	6.0	8.0	7.0	6.8	9.6	7.1
Unit weight (lb/ft^3)	144.5	145.3	142.3	141.1	143.9	139.3	143.7

*2,570 lb Type III and 480 lb Type I cement.

Results for the small test section are given in Table 7.27. Although the slump was acceptable, the air content was low at 3%. Therefore, the dosage of the AEA was increased from 4.5 to 5 oz/cwt (2.94 to 3.26 mL/kg). Two noninsulated cylinders were tested at 5.5 hours after placement of the concrete. Their measured strengths averaged 4,800 psi (33.1 MPa). This level of strength exceeds the requirement for the VES mix and nearly reaches the HES requirement in one-fourth the time. There is no explanation for why the concrete strength of noninsulated cylinders would be so high. It may be that there was more cement in the mixture than realized, although there is no proof that this is true.

7.7.5.1 Fresh Concrete Properties

The fresh properties for all truckloads of concrete are given in Table 7.28. Slumps ranged from 2.5 to 6.0 in. (63 to 150 mm), averaging about 4.5 in. (113 mm). The air contents ranged from 5.5% to 9.6%, averaging about 7%. Compared to the Illinois installation, the slumps in the Arkansas installation were higher but the air contents lower, on average.

With the increased dosage of AEA, the slump and air contents for the concrete in Truck 1 were much higher than those for the small test section. As a result, the HRWR dosage rate was reduced from 18 to 16 oz/cwt (11.75 to 10.44 mL/kg). The AEA remained at 5 oz/cwt (3.26 ML/kg). These dosage rates were used for all other truckloads of concrete.

Even with the above-noted change in the HRWR dosage factored in, there appears to be a pattern with regard to the magnitude of the slump and air content for each truckload of concrete. Beginning with the first load, every other load has a high measured slump and air content. Note that only two ready-mix trucks were used for this job. Also, a few loads of concrete were produced for other jobs prior to the start of this work. Based on this information and on the fact that the material quantities are fairly consistent for each truckload, it appears that additional water may be present in the concrete for Trucks 1, 3, and 5. One explanation for the additional water may have been that the wash water from the previous batch of concrete was not fully discharged.

Figures 7.8 and 7.9 present the temperature histories for the insulated and noninsulated patches, respectively, and their companion cylinders. As in New York, the mid-depth thermocouple in the insulated patch malfunctioned and no data were recorded. However, the temperature history for the companion cylinder follows a path similar to what one would expect for the mid-depth of the slab. For the noninsulated patch, the behavior is similar to the insulated patch, except that the mid-depth temperature is as high or higher than at the surface of the pavement. This makes sense if one considers that heat is being lost at the surface when there is no insulation. However, even though the companion cylinder had its top surface exposed to the environment as well, its temperature remained about 5°F below the mid-depth of the slab throughout the period during which data were recorded. This may suggest that the curing box detailed in Figure 7.2 is not applicable to noninsulated patches.

7.7.5.2 Hardened Concrete Properties

Table 7.29 through 7.32 present data from early-age cylinder testing, long-term cylinder testing, and coring, respectively. Cylinders cast for early-age strength determination were transported to a district office for testing. All other strength tests were conducted at the Little Rock office.

Table 7.29 shows the results from early-age testing. The average strength at 6 hours varies considerably among the various truck loads of concrete. The strength of the concrete in Trucks 1 and 3 was about 2,000 psi (14 MPa) while the strength of the concrete in Truck 2 was 3,540 psi (24.8 MPa). Recall that the same driver delivered truckloads 1 and 3.

Overall, the early strengths measured in Arkansas are much higher than in any of the previous states. All three truckloads of concrete could be said to have met the strength requirements of the VES mix. In addition, the concrete from Truck 2 gained more than 80% of the required 1-day strength for the HES mix in just 7 hours.

Table 7.30 provides data from the long-term testing program. Cylinder-to-cylinder variation in strength is much less in Arkansas than it was in Illinois. In addition, the results do not indicate any problems with the test methodology used.

Comparing the results for Trucks 2 and 5, the insulated concrete continues to have a higher measured strength than the noninsulated concrete. However, the difference between the average strengths for these two trucks has been cut in half during the period from 1 day to 6 months. Interestingly, the strengths of the concrete from all other trucks, insulated and noninsulated, are somewhat similar. Strength levels do not appear to level off for either patch until after the 28-day tests. Results from tests on lab-cured cylinders are given in Table 7.31. Generally, the

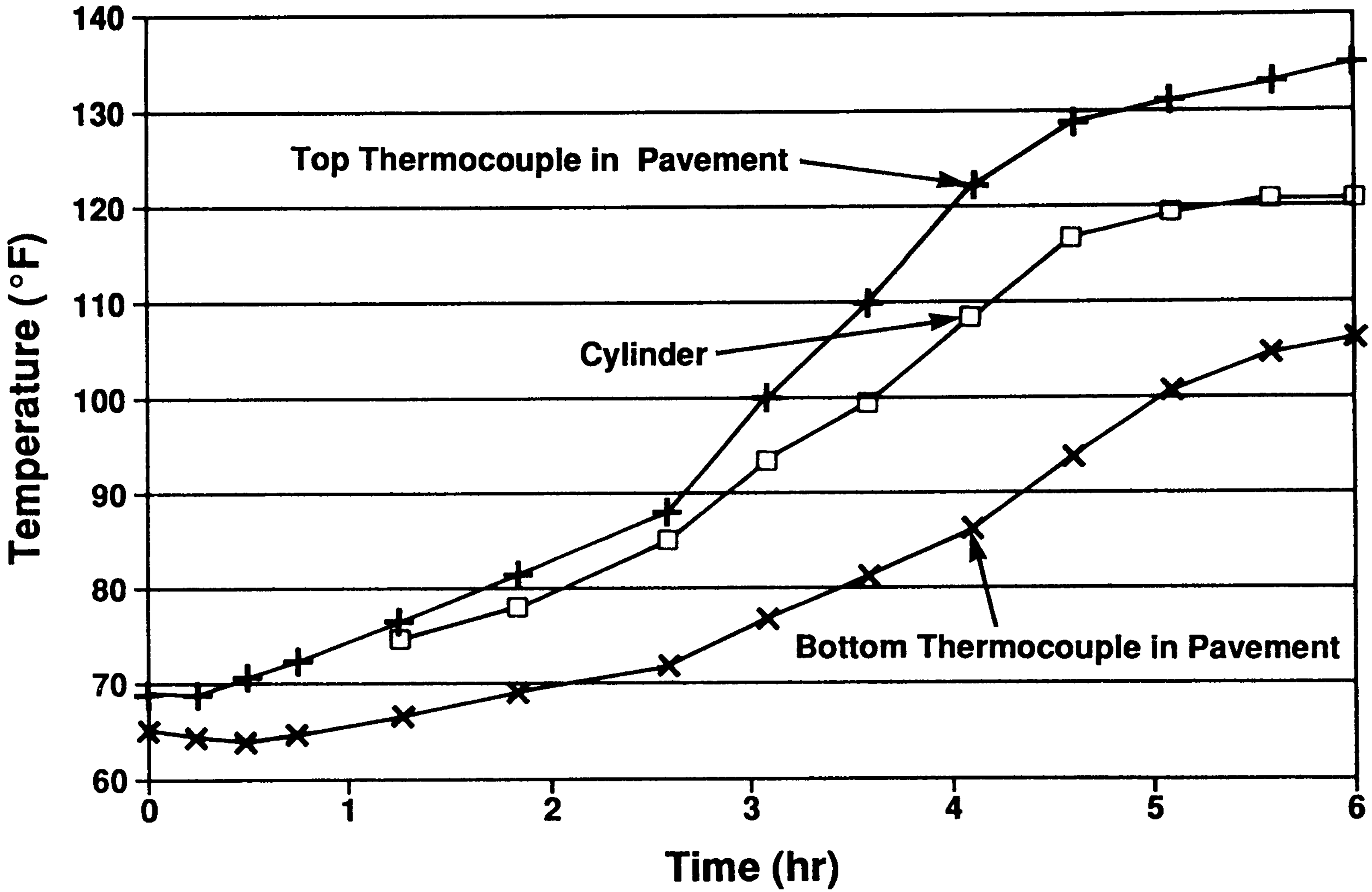

Figure 7.8 Truck 2 concrete temperature data (insulated) — Arkansas

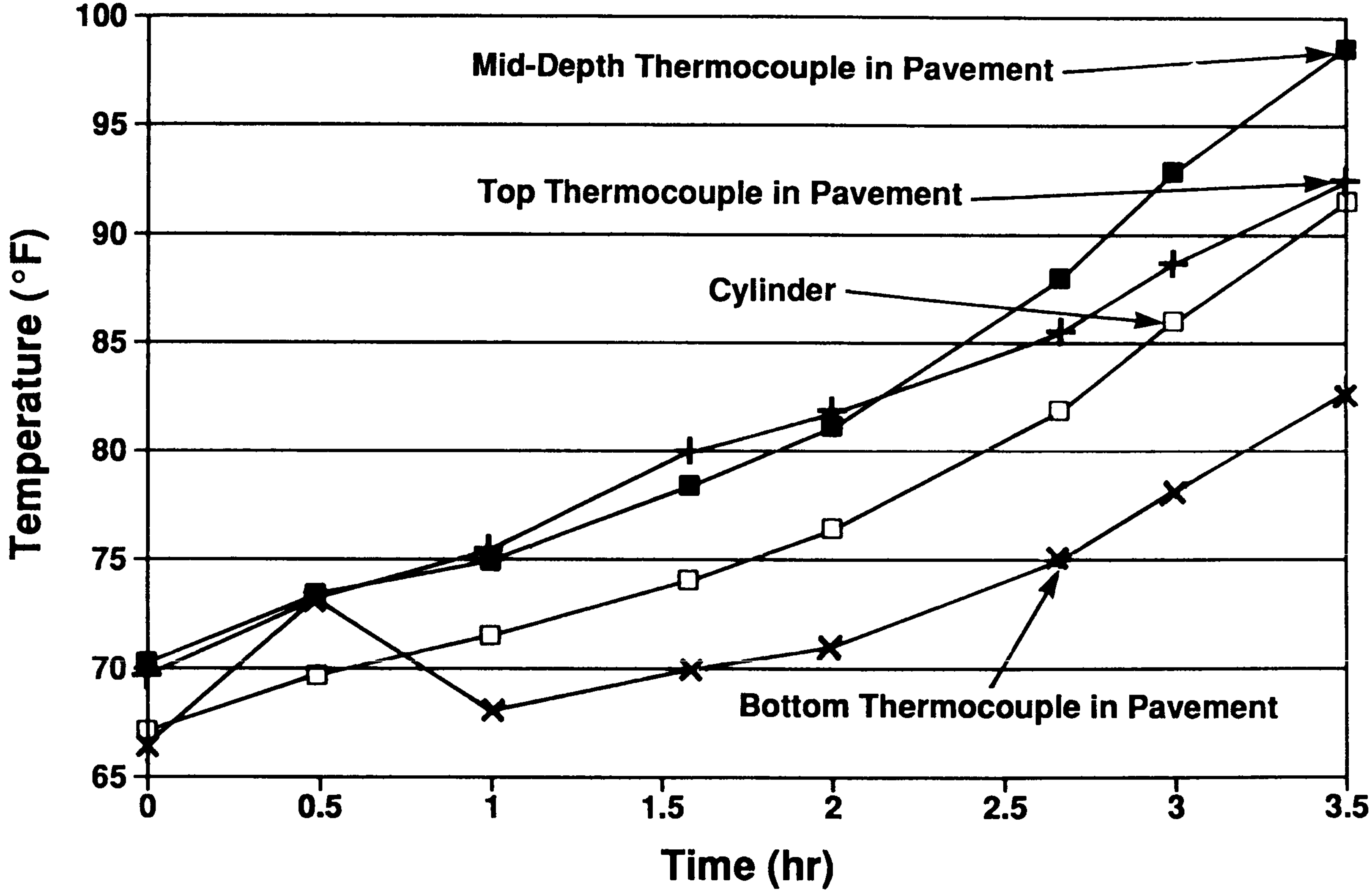

Figure 7.9 Truck 5 concrete temperature data (noninsulated) — Arkansas

Table 7.29 Early strength of 4 × 8-in. cylinders (psi) — Arkansas

Test Date	Test Age	Truck 1	Truck 2	Truck 3
11/6/91	4 Hours		1,230	
			1,390	
	5 Hours		2,900	
			2,940	
	6 Hours	1,670	3,540	2,110
		2,310	3,540	1,990
	7 Hours		4,140	
			3,990	

Table 7.30 Long-term strength of 4 × 8-in. cylinders (psi) — Arkansas

Test Date	Test Age	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	Truck 6
11/7/91	1 Day		5,710		4,050	4,440	3,360
			6,110		4,080	4,680	3,450
11/13/91	7 Days	6,410	7,400	5,970	6,480	6,640	5,690
		6,800	7,840	6,050	6,610	5,970	5,870
			7,360			5,970	
12/4/91	28 Days	8,870	9,110	7,560	8,320	8,360	6,880
		9,070	9,270	7,520	7,960	8,560	7,000
5/7/92	6 Months		10,980			10,340	
			10,500			9,710	
11/9/92	12 Months		12,530			11,420	
			12,730			10,940	
5/93	18 Months		8,440			10,740	
			10,820			10,580	

Table 7.31 28-Day strength of laboratory-cured cylinders (psi) — Arkansas

Cylinder Size	Truck 2	Truck 5
4 × 8 in.	10,310	10,050
	10,380	8,760
6 × 12 in.	9,820	8,860
	9,870	8,740

Table 7.32 Compressive strength of 4-in. cores (psi) — Arkansas

Test Date	Test Age	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	Truck 6
12/5/91	28 Days	7,600	7,600	6,490	7.040	6,740	6,640
			8,160			7,420	
			9,030			7,360	
6/5/92	6 Months	9,750	9,030	7,680	7.880	10,740	7,640
			10,150			8,440	
			9,670			9,350	
11/9/92	12 Months	10,660	10,160	8,010	9,060	10,600	8,030
			10,700			9,360	
			10,520			8,730	
5/93	18 Months	10,230	8,750	8,040	8,240	8,790	8,040
			9,550			9,150	
			9,590			9,150	

strength from these tests correlates better with the noninsulated concrete than with the insulated concrete. A 6 x 12-in. (150 x 300-mm) cylinder cast from the concrete from Truck 2 by AHTD personnel, lab-cured, and tested at 7 days measured 7,300 psi (51.1 MPa). This strength is in line with the field-cured cylinders for Truck 2. No lab-cured cylinders were tested at 7 days, so no comparisons can be made.

Core data are fairly consistent with the cylinder data. On average, cores taken from Truck 2 show a higher measured strength than those from Truck 5. Cores from all other trucks have roughly the same level of strength. All cores indicate a strength gain between 28 days and 6 months.

Beams were cast with concrete from Trucks 2 and 5. These beams were tested under third-point loading at 1, 7, and 28 days of age. Results are given in Table 7.33. Strength variation between beams is low for some tests and high for others. This makes it difficult to extract any meaning from the data. Beams, 6 x 6 in. (150 x 150 mm), cast by AHTD personnel with concrete from Truck 2, had measured strengths of 925 psi (6.4 MPa) at 7 days and 1,240 psi (8.5 MPa) at 28 days. Both results are far greater than that found for the 4 x 4-in. (100 x 100-mm) beams cast by the research staff.

Table 7.33 Flexural strength of 4 × 4-in. beams (psi) — Arkansas

Test Date	Test Age	Truck 2	Truck 5
11/7/91	1 Day	590 470	440 440
11/13/91	7 Days	690 530	610 450
12/4/91	28 Days	1,050 780	730 740

7.7.6 Follow-up Site Visit

A visit was made to the Arkansas installation on February 26, 1993. There was no evidence of distress in any of the six panels of hig performance concrete. The concrete was in very good condition.

A potential trouble spot may exist along the inside edge of both patches. During construction of the patches, a portion of the existing asphalt shoulders was removed so that the formwork for the concrete patches could be properly positioned. After the formwork had been removed, an asphalt mixture was used to fill the gap between the concrete and the original shoulder. Currently, there is about a 2 in. (50 mm) difference between the top of the concrete patch and the top of the asphalt shoulder. The concrete is higher,so ponding may be occuring along the edge of the patches and water may be infiltrating the concrete. In addition, the asphalt immediately adjacent to the patch was not well compacted. This may eventually lead to deterioration of the concrete along the outside edge of the patch.

7.8 Test Pavement — Nebraska

7.8.1 Site Description

The fifth field installation was constructed in Nebraska. The site is northwest of Norfolk, between the towns of Osmond and Plainview. A patch 96 ft (29.3 m) long was constructed on July 15, 1992 in the eastbound lane of US Highway 20 at mile marker 361 (Station 435).

US 20 was built in 1957. It is a small rural road in northern Nebraska that cuts across farm land and through a number of small towns. It has two 11 ft (3.4 m) lanes,giving a driving surface 22 ft (6.8 m) wide. The pavement is 8 in. (200 mm) of plain portland cement concrete over a 4 in. (100 mm) granular base and a silty clay subgrade. Joints are evenly spaced at 16 ft (4.9 m). Earth shoulders existed in the area of the site when the installation was constructed.

The installation was part of a large repair and rehabilitation contract. Failed concrete pavement sections were being replaced. An asphalt shoulder was being added in areas. In addition, at least two bridges were being replaced between Osmond and the site. Upon later inspection of one of the bridge sites, pieces of construction debris clearly showed evidence of alkali-aggregate reactivity. Two pieces of this material were retrieved by the research staff.

The section of roadway selected for the installation was badly cracked. A longitudinal crack extended the length of six panels, or 96 ft (29.3 m). In the immediate vicinity of the installation, the concrete seemed to be in good condition.

Traffic at the site is very light. Data from the Nebraska Department of Roads (NDR) indicates 1,500 vehicles per day and 20% trucks in the area. There are, however, occasional heavy loads from tractor-trailers and farm implements. The climatic exposure of the pavement can be described as wet with a high potential for freezing-thawing cycles, about 60 per year according to NDR. De-icers are typically used; according to NDR, as many as 15 applications per year can be expected.

7.8.2 Patch Description and Preparation

One section of concrete 96 ft (29.3 m) long was removed and replaced with high performance concrete. This represents two 48 ft (14.6 m) patches built end-to-end. Each patch contains three 16 ft (4.9 m) sections. The individual patches are slightly longer but narrower than a typical patch. However, in most other respects, the patches can be considered typical, as described in Section 7.3.

The existing concrete was removed on the morning of the concrete placement. Because the site is only a two-lane highway, safety concerns did not permit overnight lane closures. A large-diameter saw cut the pavement into panels 16 ft (4.9 m) long. The concrete was then removed using a large loader and a small bobcat loader. The large loader traveled across the patch area as it removed a section of pavement; thus a significant amount of damage was done to the subbase.

The subbase was leveled out using sand. The sand was manually screeded and then compacted with a vibrating plate.

Transverse joints are located on 16 ft (4.9 m) centers. Epoxy-coated dowel bars were placed at all transverse joints. There were no caps on the dowel bars. Dowel bars were grouted into the existing concrete and all bars were greased. Dowel baskets used for the internal joints were poorly aligned, as were the dowels themselves. This may lead to joint problems in the future.

Thermocouples were placed in each section of both patches. An additional thermocouple was placed at mid-depth near the outside edge of the center section of both patches. The purpose of this thermocouple was to determine if there was much heat loss at the outside edge of the pavement. Thermocouples for a maturity meter were also installed in the insulated patch by NDR personnel. Continuous monitoring of the temperature history in the insulated patch, and of a corresponding cylinder in a curing box, had been planned as part of an effort to examine the applicability of the maturity method to high performance concrete. However, the data acquisition system malfunctioned and no useful data were recorded. The power supply for the maturity meter also malfunctioned.

In addition to the experimental patch, a trial section 16 ft 4 in. (5 m) long, 11 ft (3.4 m) wide, and 11 in. (275 mm) deep was constructed on July 14, 1991.

7.8.3 Materials and Proportions

The HES mixture described in Table 7.1 was used for this installation. Data on the coarse aggregate are given in Table 7.34. The dosage rates for the HRWR and AEA were initially set at 18 oz/cwt and 5 oz/cwt (11.75 and 3.26 mL/kg), respectively.

Proportions for the trial patch are given in Table 7.35. The slump for this mixture was very high. Based on appearance, there did not seem to be much coarse aggregate in the mixture. In fact, one laborer was overheard making the following comment to a co-worker: "What's so different about this stuff." The significance of that remark is that, due to alkali reactivity problems with the coarse aggregate in the region, the state limits the coarse aggregate fraction of a concrete mixture to 30% of the total aggregate volume. Thus, this laborer noticed that there wasn't much rock in the mixture. Cylinders cast and tested for early-age strength confirmed that there was little rock in the mixture. It appears that in batching, the weights of the coarse and fine aggregates were reversed from the proportions intended. A high sand content would account for the more flowing and easy finishing characteristics of the trial mixture. Thus, it was felt that there was an error in batching the concrete for the trial section.

Table 7.36 gives the results from on-site testing of cylinders from this concrete. The patch and cylinders were insulated. The strength development of this concrete was very good. These cylinders met the strength requirement of the VES mixture at 4 hours and had about 60% of the HES strength requirement at 5 hours.

Table 7.34 Aggregate gradation and material properties — Nebraska

Sieve Size	Percent Passing	
	Coarse	Fine
1 in.	100.0	
1/2 in.	80.0	
1/4 in.	45.0	
3/8 in.	30.0	
No. 4	9.0	89.0
No. 8	5.0	67.0
No. 16	4.0	48.0
No. 30		33.0
No. 50		18.0
No. 100		3.0
Specific gravity	2.65	2.61
Absorption (%)	1.51	0.82

1 in. = 25 mm

Table 7.35 Batch quantity for test patch — Nebraska

Material	Total Quantity Per Batch
Batch size (yd^3)	6.5
Cement (Type III) (lb)	5,673
Water (lb)	1,488
Coarse aggregate (lb)	11,120
Fine aggregate (lb)	7,070
HRWR (WRDA-19) (oz)	1,020
AEA (Daravair) (oz)	284
Calcium nitrite (DCI) (gal)	26
Slump (in.)	7.75
Air content (%)	6.1
Unit weight (lb/ft^3)	144.0

Table 7.36 Compressive strength of 4 x 8-in. cylinders (psi) — Nebraska

Time	Cylinder 1	Cylinder 2
4 Hours	2,550	2,390
4.5 Hours	2,790	2,790
5 Hours	2,870	2,980

Based on the apparent batching error, a 1.5 yd^3 (1.1 m^3) mixture was cast the following morning for verification (see Table 7.37). Extra care was taken to ensure that the coarse and fine aggregates were batched correctly. Additional water was added to compensate for the DCI, which was not added to this mixture. The concrete was taken across the street from the plant for inspection and placement in forms for other uses. This small batch of concrete more closely resembled the material used in the laboratory and at other job sites. It was stiff and sticky. Therefore, it was confirmed that an error had occurred the previous day.

Table 7.37 Trial batch quantities — Nebraska

Material	Total Quantity Per Batch
Batch size (yd^3)	1.5
Cement (Type III) (lb)	1,305
Water (lb)	311
Coarse aggregate (lb)	2,571
Fine aggregate (lb)	1,598
HRWR (WRDA-19) (oz)	236
AEA (Daravair) (oz)	65
Calcium nitrite (DCI) (gal)	—

A decision was made at that time to intentionally invert the coarse and fine aggregate quantities for the work. This was done for two reasons. First, except for a rather high slump, the concrete produced the previous day seemed good. The early-age strengths were excellent and the workability was acceptable. Second, a prime objective of the field work was to determine whether high perperformance concrete could be produced using local materials and methods. Thus, if local requirements dictate a 30% maximum coarse aggregate fraction, the work should fall within this restriction. It was felt that this was within the sprit and intent of this research project. Therefore, all batches of concrete were produced with the batch weights of the coarse and fine aggregates reversed.

Note that although the batch weights were reversed, no correction was made to these weights for differences in specific gravity. Table 7.34 provides the specific gravities for the Nebraska aggregate. Had the weights been corrected for the specific gravity of the aggregates, 130 fewer lbs (59 kg) of fine aggregate and 80 more lbs (36.3 kg) of coarse aggregate would have been batched for Trucks 1 through 5. These weights may seem large, but for a 5.5 yd^3 (4.2 m^3) batch, this amounts to a difference of only 1.4% for both the fine and coarse aggregate. This percentage is considered to be insignificant.

7.8.4 Batching, Placing, and Curing

Trucks were batched with 5.5 yd^3 (4.2 m^3) of concrete for this installation. A smaller batch size was used in Nebraska due to the size of the patches. Although longer than in other states, the width and depth of the patch were much less. The last truck was a 6.0 yd^3 (4.6 m^3) batch to ensure that enough material was available to complete the patch.

Dry batching was done at a small batch plant located in Osmond. Thus, travel time to the job site was about 5 minutes. This could have increased to 10 minutes if delays were encountered due to other construction along US 20. The split addition of materials was necessary due to scale capacity. Otherwise, there were no notable problems with batching.

Table 7.38 presents the batch weights for all truckloads produced. The dosage rate for the HRWR was very high in Nebraska. The first truckload of material was inspected at the batch plant by the research staff prior to being sent to the job site. The slump at that time was very low. As a result, additional HRWR was added to the mix. A dosage rate of 24 oz/cwt (15.66 mL/kg) was required to achieve a 6-in. (150-mm) slump at the batch plant. The high slump was

Table 7.38 Actual batch weights and fresh concrete properties — Nebraska

Material	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	Truck 6
Batch Size (yd^3)	5.5	5.5	5.5	5.5	5.5	6.0
Cement (Type III) (lb)	4,813	4,809	4,849	4,809	4,825	5,233
Water (lb)	1,106	1,208	1,200	1,130	1,233	1,270
Coarse aggregate (lb)	5,750	5,815	5,855	5,805	5,830	6,320
Fine aggregate (lb)	9,610	9,615	9,615	9,600	9,620	10,500
HRWR (WRDA-19) (oz)	1,152	960	1,024	960	960	1,088
AEA (Daravair) (oz)	240	240	230	230	215	246
Calcium nitrite (DCI) (gal)	22	22	22	22	22	22
Slump (in.)	4.5	3.0	6.6	4.5	1.5	7.0
Air Content (%)	11.5	6.6	14.0	8.6	6.2	16.0
Unit weight (lb/ft^3)	133.4	139.7	133.1	136.8	142.0	123.2

used to compensate for slump loss at the job site. All remaining truckloads had an HRWR dosage rate of 20 to 21 oz/cwt (13.05 to 13.70 mL/kg). The AEA dosage ranged from 4.4 to 5.0 oz/cwt (2.94 to 3.26 mL/kg).

Construction of the installation began at the east end of the patch and proceeded in a westerly direction. The noninsulated patch was constructed first. This is the opposite of what occurred at the other installations. The insulated patch was constructed second because early-age cylinders were to be tested on site. Therefore, additional time to transport the cylinders to a testing facility was not needed. Concrete from Trucks 1 through 3 was used for the noninsulated patch; concrete from Trucks 4 through 6 was used for the insulated patch.

When the truck arrived at the job site, the calcium nitrite solution was pumped into the truck. The volume was controlled with a metering device. The concrete was mixed, discharged, andconsolidated with a spud vibrator and mechanical screed. Curing compound was applied as soon as finishing was complete. Insulation was applied as soon as the concrete had sufficient set — sometime around 2 p.m. The insulation was removed from the patch at 7:30 p.m., thus the patch was insulated for roughly 5.5 hours. Joints were sawed in the noninsulated patch around 7 p.m. When the insulation was removed from the insulated patch, joints were then sawed for that patch.

Once all joints were sawed and the area was cleared of all construction debris, the patch was opened to traffic. This was around 8 p.m. on July 15.

7.8.5 Results and Discussions

Properties of the fresh concrete were determined by NDR personnel. Cylinder and beam specimens were cast by the research staff. Three cylinders and two beams were cast for each test age.

All specimens were cured on site in polystyrene curing boxes. Two 4 x 8-in. (100 x 200-mm) and two 6 x 12-in. (150 x 300-mm) cylinders, cast with concrete from Truck 2 for each patch, were cured in air for 24 hours and then brought to NDR Lincoln office for standard curing until testing. Cylinders cast for early-age testing were tested on site with a portable compression machine.

Results for the trial batch are given in Table 7.35. A slump of nearly 8 in. (200 mm) was too high. The air content was at an acceptable level. It is believed that the high slump was partly due to the high proportion of fine material in the mixture. Although slump and air content were not determined for the 1.5 yd^3 (1.1 m^3) verification batch, visual inspection indicated a much lower slump. Strength data presented in Table 7.36 show that the strength of the trial batch reached almost 3,000 psi (21 MPa) in 5 hours. This was high relative to the other field installations. These high strengths gave the additional reason for the decision to reverse the aggregate proportions of the concrete used for actual construction of the installation.

7.8.5.1 Fresh Concrete Properties

The fresh concrete properties for all truckloads are presented in Table 7.38. Even with a batching error, production and placement of the trial batch went fairly well. This was not the case during construction of the actual field installation. As can be seen from Table 7.38, the slumps and air contents varied considerably. Slump ranged from 1.5 to 7.0 in. (38 to 175 mm) and the air content varied from 6.2% to 16.0%.

Changes in the admixture dosages, for minor adjustments of the mixture, did not produce the expected results. For example, the dosage of AEA in Truck 2 was 5 oz/cwt (3.26 mL/kg) and the HRWR was at 20 oz/cwt (13.05 mL/kg). The concrete had an air content of 6.6% and a slump of 3 in. (75 mm). The AEA dosage was reduced to 4.7 oz/cwt (3.07 mL/kg) and the HRWR was increased to 21 oz/cwt (13.7 mL/kg) in order to lower the air content and raise the slump slightly. The result was an air content of 14.0% and a slump of 6.5 in. (163 mm) for Truck 3. The changes in slump and air content were not consistent with the changes in the dosages. The difference between the AEA dosages for Trucks 4 and 5 was about 0.5 oz/cwt (3.26 mL/kg) with no change in the HRWR dosage. The result was a drop in slump of 3 in. (75 mm) and almost 2.5% in air content. Again, these changes were not consistent with the changes in the admixture dosages.

These mixtures lost slump more rapidly than any of the other mixtures previously used. In fact, the last two truckloads had a significant amount of water added at the site in order to discharge the concrete from the truck. The last section of the insulated patch ultimately failed from low strength. This low strength was likely due to the amount of water that had to be added at the job site. Even then, about 1 yd^3 (0.8 m^3) of concrete was left in the truck and finally dumped next to the site because it had become too stiff to work with.

Figures 7.10 and 7.11 present the temperature histories for the two patches and their companion cylinders. For the insulated patch (Figure 7.10), the top and mid-depth thermocouple were connected to a data acquisition system that failed to record properly. The bottom and edge (mid-depth) thermocouples were monitored manually. As at the other sites, the cylinder temperature history was similar to that at the mid-depth of the patch. For the noninsulated patch (Figure 7.11), as in Arkansas, the cylinder temperature was as high as, or higher than the temperature at the mid-depth of the patch. Both temperatures are typically higher than at the surface or base of the patch. As noted in the section on the Arkansas installation, when left uncovered, the patch loses heat to the environment and so the mid-depth will have a higher temperature. In contrast to Arkansas, in Nebraska, the cylinder temperature was more in line with the mid-depth temperature.

7.8.5.2 Hardened Concrete Properties

Tables 7.39 through 7.43 present data from cylinder and beam tests. Early-age cylinders were tested on site with a portable compression machine. Long-term cylinder and core testing was conducted both on site and at the Lincoln office of NDR. Neoprene pads were used for the cylinder and core tests.

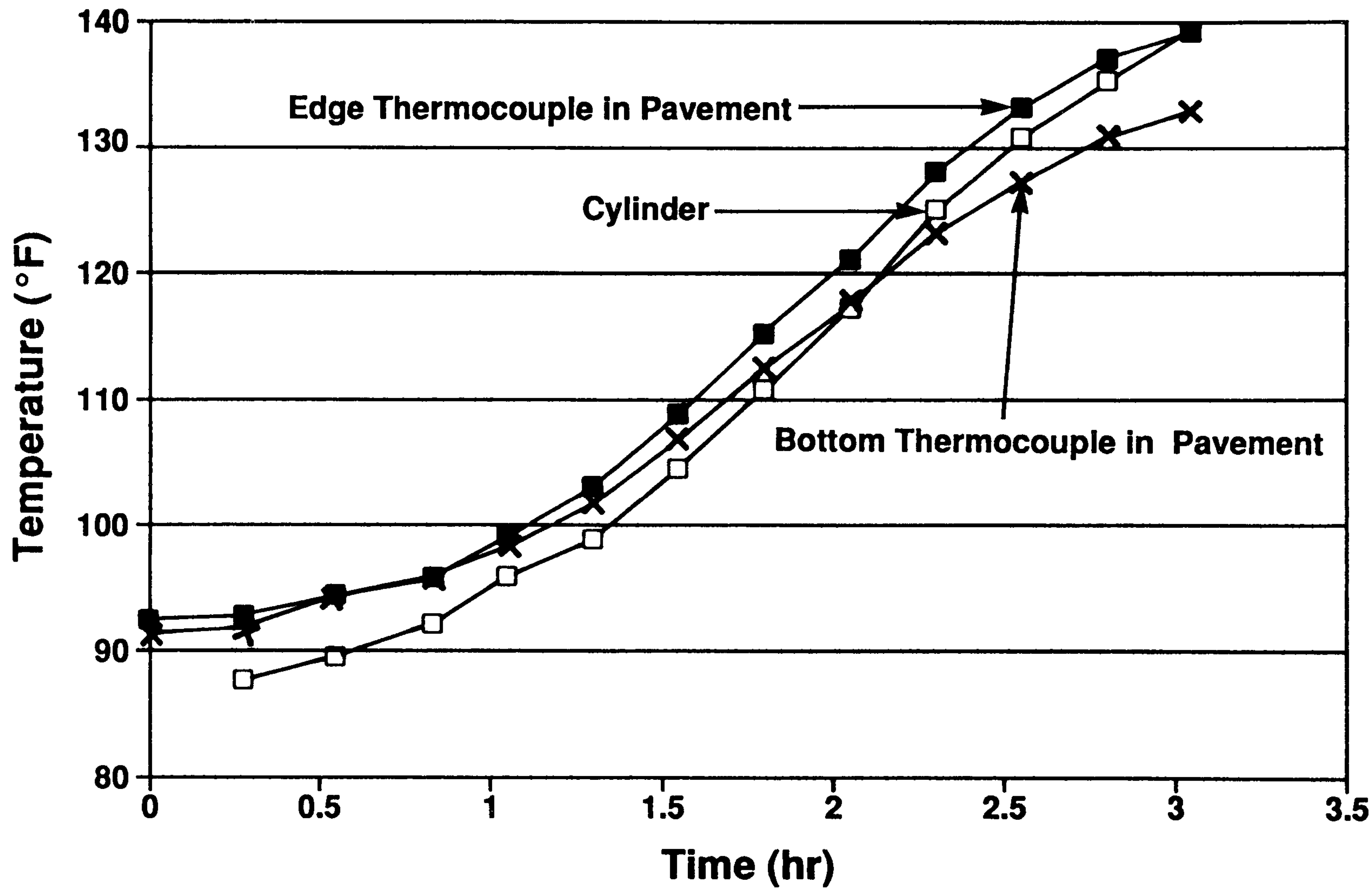

Figure 7.10 Truck 2 concrete temperature data (insulated) — Nebraska

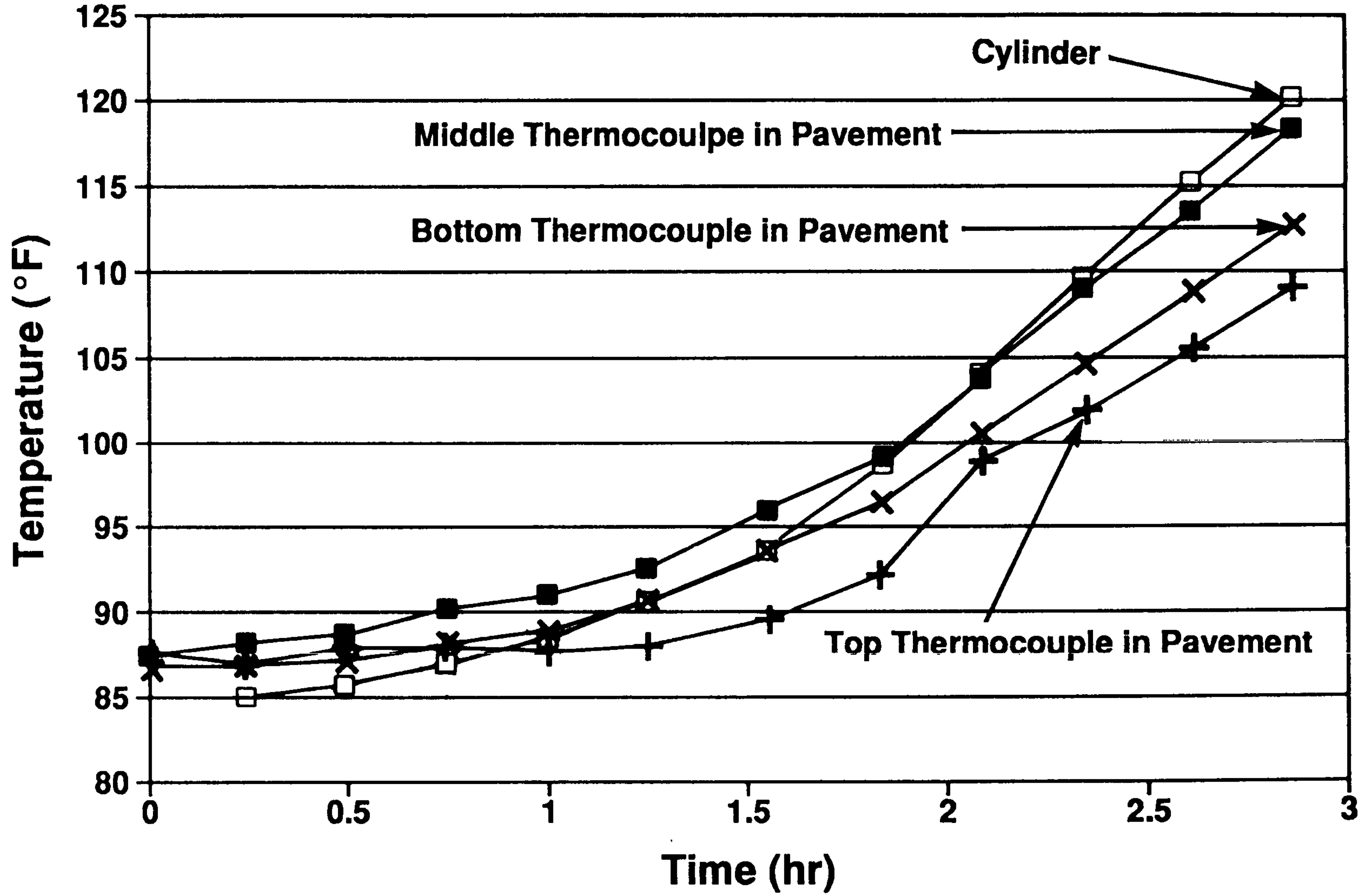

Figure 7.11 Truck 5 concrete temperature data (noninsulated) — Nebraska

Table 7.39 Early strength of 4 × 8-in. cylinders (psi) — Nebraska

Test Date	Test Age	Truck 4	Truck 5	Truck 6
7/15/92	4 Hours		3,020	
			2,980	
			3,140	
	5 Hours		3,340	
			3,300	
			3,500	
	6 Hours	2,940	3,660	760
		3,060	3,660	800
		3,060	3,780	800
	7 Hours		4,060	
			4,180	

Table 7.40 Long-term compressive strength of 4 × 8-in. cylinders (psi) — Nebraska

Test Date	Test Age	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	Truck 6
7/16/92	1 Day	3,780	4,340	2,590	4,380	4,890	1,790
		3,740	4,500	2,670			
		3,580	4,580	2,510			
7/22/92	7 Days	5,050	5,770	3,660	5,250	6,450	2,270
		5,170	6,050	3,660	5,370	6,490	2,230
		5,090	5,570	3,500	5,410	6,050	2,470
8/12/92	28 Days	5,610	6,560	3,900	5,890	6,920	2,830
		5,050	6,450	4,060	6,050	6,450	2,750
		5,530	6,610	3,980	5,890	6,760	2,750
1/15/93	6 Months		7,400			8,320	
			7,800			8,160	
			8,120				

Table 7.41 28-Day strength of laboratory-cured cylinders (psi) — Nebraska

Cylinder Size	Truck 2	Truck 5
4 x 8 in.	6,610 6,660 6,450	6,450 6,450 6,230
6 x 12 in.		6,550 6,520 6,610

Table 7.42 Compressive strength of 4-in. cores (psi) — Nebraska

Test Date	Test Age	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	Truck 6
8/12/92	28 Days	4,720	5,560 5,470 6,000	3,210	5,160	6,200 6,180 6,060	Patch Removed
1/15/93	6 Months	5,060	5,430 6,060 5,980	3,130	4,270	5,040 5,090 6,520	

Table 7.43 Flexural strength of 4 x 4-in. beams (psi) — Nebraska

Test Date	Test Age	Truck 2	Truck 5
7/22/92	7 Days	530 360 360	320 320 320
8/12/92	28 Days	310 300 310	380 350 290

Table 7.39 presents the early-strength data. These data show the excellent strength development for the concrete from Truck 5. The concrete clearly exceeded the strength requirements of the VES mix. These strengths are slightly higher than for Truck 2 in Arkansas at 4 and 5 hours, but the strength levels are similar at 6 and 7 hours. The measured strength of the concrete from Truck 4 is lower than that of Truck 5. This can be attributed to the higher air content for Truck 4. The concrete in Truck 6 had very low early-age strength. This load and the first truckload in New York had similar early-age strength levels.

The relatively low strength of the concrete from Truck 6 continued through 28 days. Prior to 28-day testing, the last section of the insulated patch had cracked. Based on its low strength and crack development, it was decided that the last section would be replaced. This section of concrete was removed and replaced with the state's standard patch mix. At this time, this is the only section at any of the field sites that has been replaced.

Table 7.40 presents the long-term strength data for the Nebraska installation. Strength variation among cylinders is low in the Nebraska testing. Results are consistent with the use of insulation and the various air contents of the concrete as shown in Table 7.38. The insulated sections have higher strengths through 6-month testing. In addition, batches with high air contents have lower strengths regardless of whether the concrete was insulated. Neither patch developed 1-day strengths meeting the requirement of the HES mix, although Truck 5 came close.

Data for the laboratory-cured cylinders are given in Table 7.41. The sample of material taken from Truck 2 turned out to be too small to cast any 6 x 12-in. (150 x 300-mm) cylinders, so only 4 x 8-in. (100 x 200-mm) cylinders were produced. At 28 days, the strength of the laboratory-cured cylinders is close to that of the field-cured cylinders for Truck 2. However, for Truck 5, the field-cured cylinders are slightly stronger. This may be because the laboratory-cured cylinders were initially air-cured and then transported to the lab. The field-cured cylinders were insulated for 24 hours before being placed in the ground.

Core strengths are quite a bit lower than the strengths of field cured cylinders. These data are given in Table 7.42. Not only are the strengths lower, they appear to be dropping. For Truck 5, the 28-day average core strength was about 6,150 psi (43 MPa). At 6 months the average strength was 5,530 psi (38.7 MPa) if the third core is considered and 5,070 psi (35 MPa) if the third core is ignored because of its high strength. In any case, the drop in strength is significant. For Truck 2, the 28-day and 6-month average core strengths are 5,680 psi (40 MPa) and 5,820 psi (41 MPa), respectively. This is only a slight increase. It will be necessary to wait for 12-month data before any firm conclusion can be made regarding this expected trend.

Data from the testing of 4 in. (100 mm) beams is provided in Table 7.43. As in Illinois and Arkansas, the data do not seem to provide much valuable information.

7.8.6 Follow-up Site Visit

The Nebraska installation was revisited in late January 1993. On inspection of the site, three transverse cracks were discovered. These cracks were first reported by NDR personnel when 6-month testing was conducted. Cracks are present in two of the three noninsulated sections and one of the insulated sections (see Figure 7.12). All three cracks appeared to be wider than 1/16 in. (1.6 mm), although no measurement was actually taken. They also seemed to be located roughly near the center of the patch. There were no signs of distress in any of the other four sections.

Interestingly, a similar cracking pattern exists at other patches in the vicinity of the installation. Those patches were constructed with Nebraska's standard patch mixture around the same time as the field installation. It may be that these cracks developed because a lack of subgrade support. Recall that in removing the old concrete, a large loader traveled across the subbase to remove the next panel, and the subbase was noticeably disturbed in the process. If this material was not adequately compacted, it could fail to provide the necessary support to the pavement.

All joints at the Nebraska installation appeared to be in good condition, as did the remainder of both patches.

7.9 Discussion

As a result of five field test of high performance concrete, much has been learned about this material and what it takes to produce and place it properly. To use this material successfully, certain aspects of the construction process require special attention. Below is a brief discussion of what might be the three most important lessons learned from the field installations. This discussion is meant only to provide some guidance to a potential user of this innovative material.

1. Thorough mixing of this concrete is critical. This is true even with central mixing. Worn blades, fin build-up, or minimum drum rotation speed will hinder effective mixing of the constituent materials. Chain-driven drums have proven susceptible to mixing problems with dry-batched high performance concrete. The staggered batching process, the low W/C, and site addition of the calcium nitrite make it important that mixing be sufficient to produce a consistent material from front to back of a giventruck.

   Generally, it is suggested that batch sizes be limited to no more than two-thirds of the truck's rated mixing capacity. This holds true for all truck mixers, even those in good operating condition. If adequate mixing is a problem, or if the condition of the trucks is questionable, it may be helpful to limit the load size to no more than one-half the rated mixing capacity. In the end it will be better to tie up more trucks and produce a good-quality material than to free trucks for other jobs and have difficulties that will last more than 1 day.

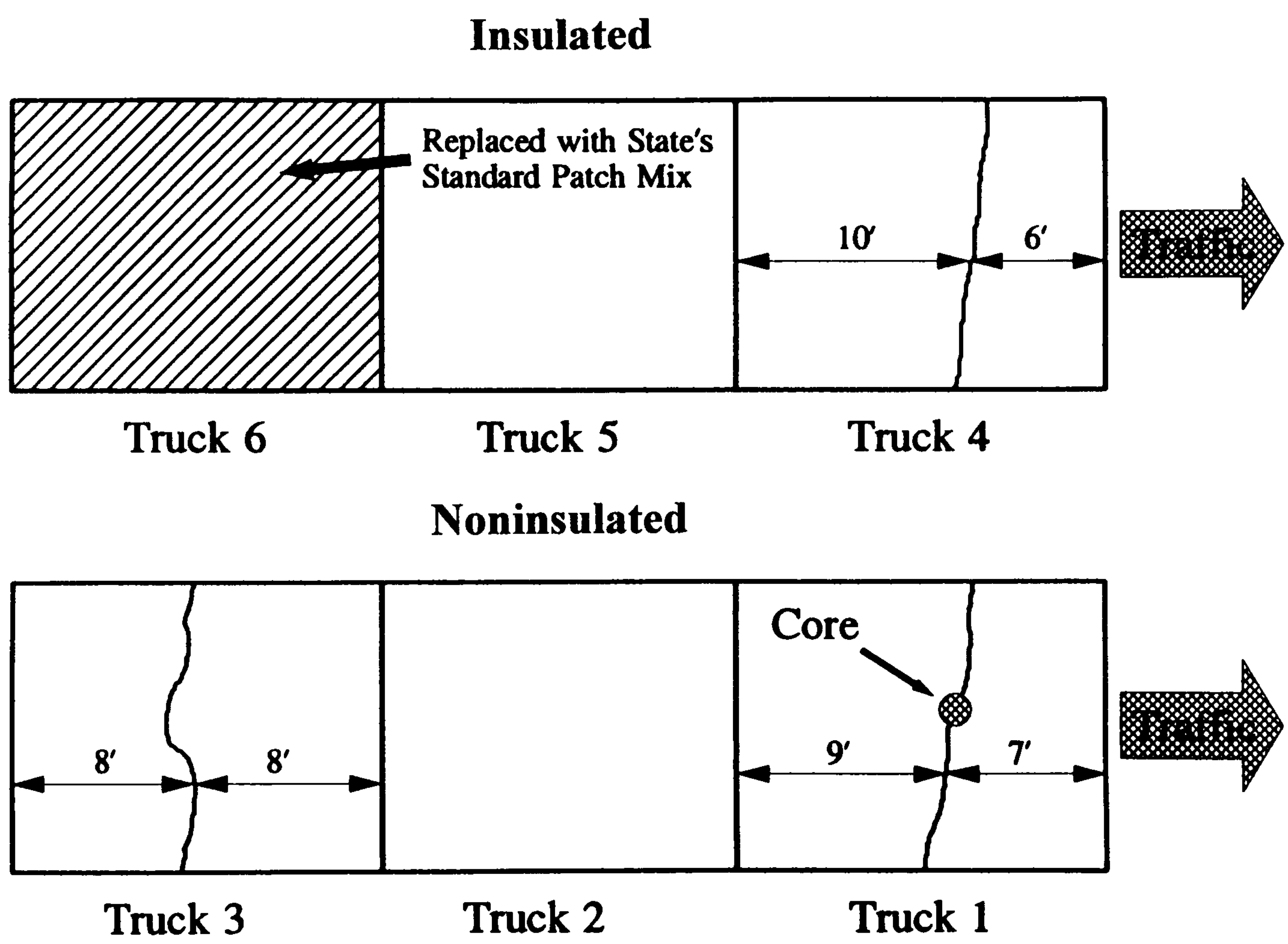

Figure 7.12 Cracks in Nebraska installation

2. A preconstruction meeting should be held with the contractor, concrete supplier (including the batch plant operator), and appropriate highway department personnel. Although recommended for any concrete construction, such a meeting is especially important when high performance concrete is used. All aspects of the project should be discussed and potential problems addressed. Topics to be considered should include the mix proportions and needed materials; the batching sequence, including the site addition of calcium nitrite; travel time and route; placing, consolidation, and finishing procedures; the stiff and sticky nature of the mixture; insulation; criteria for opening the patch to traffic; and field trial batching.

   If batch plant and contractor personnel are not included in the preconstruction meeting, it is suggested that short meetings also be held with them. As high performance concrete is relatively new to much of the country, most batch plants have not had an opportunity to produce it. Finishers have probably not worked with the concrete before either. Thus the batch plant needs to be made aware of the desire for tight control over water content. At both the plant and job site, excess water will adversely affect the performance of the high performance concrete more than it would with conventional concrete. In addition, more care must be taken in the batching process to ensure that the correct materials are batched in the proper sequence. Drivers should be made aware that delays in transporting the concrete can cause serious problems. They must also understand the need to fully discharge all wash water from the truck before it is charged with the high performance concrete mixture and the need to keep the drum in constant rotation.

   Contractor personnel should be informed of the stiff and sticky nature of the concrete. High-performance concretes usually do not flow like the mixtures commonly used in patching. Thus more effort will likely be required to place the concrete. The mixture will react well to vibration, however, a vibrator should not be used to move the concrete to its final position, as this may cause the mixture to segregate. Usual finishing practice still applies to high performance concrete, although the finishers may find the concrete somewhat difficult to work. If possible, the finishers should have a chance to work with the concrete during trial batching. This will greatly reduce problems and complaints later.

3. Laboratory and field trial batches should be produced in sufficient number to confirm the mixture proportions, batching sequence, and workability of the concrete. The basic proportions of the high performance concrete mixture will have to be modified for the physical characteristics of the local aggregates and cement. It has been found that use of ACI's proportioning guidelines (1993b) for determining aggregate quantities yields a satisfactory first approximation to the mixture proportions. The brand, type, and time of addition of the admixtures will also affect the properties and performance of the concrete.

   Laboratory batching of the constituent materials should be conducted under conditions as close as possible to those expected in the field. Slump, air content, unit weight, and temperature of the fresh concrete should be determined. A number of cylinders should be cast to evaluate the rate of strength gain and ultimate strength capacity. Curing of the cylinders should be as expected in the field. The mix proportions should be adjusted until the desired performance is achieved. At least one additional batch of a successful mix should be produced for confirmation.

After adjustments to the mixture proportions have been made in the laboratory, field trials should be conducted. The mixture almost always requires some further adjustments in the field to obtain the desired slump, air content, strength, and durability. The moisture content of the aggregate will play a significant role in adjusting the mixture proportions. The best approach to the field trials is to produce a number of small batches to confirm the mixture proportions and the time available to work the material. These batches can be used for patching, placed in temporary forms in the batch plant yard, or (for example) used as temporary working slabs on site.

With small loads adequate mixing can be ensured, waste is minimal, a single patch can be filled by a single truck, and many trials can be conducted without excessive costs. If the concrete is used for patching, discontinuous patches are best. This will eliminate the potential for any cold joints due to construction delays. In addition, work can be discontinued at just almost any time. Only after the mixture has been successfully produced in field trials should any major construction be undertaken. Between the preconstruction meeting and field trial batching, most potential problems can be addressed and remedied.

A final note on working with high performance concrete in the field concerns patience. It has been the experience of the research team that 1 full day of production is typically required to bring all participants "up to speed." Also, the learning-curve costs during this first day can be appreciable. However, at field sites where multiple days of operation were possible, subsequent work went much more smoothly.

8

Conclusions

This report has documented the results of an extensive program of laboratory studies and the experience of five field experiments on the performance of high early strength (HES) concrete, a class of high performance concrete designed for highway applications. Based on the results of this investigation, the following conclusions can be drawn:

1. Using conventional materials and equipment, but with more care than needed for conventional concrete, it is possible to produce in the laboratory, as well as in the field, HES concrete that will achieve a minimum compressive strength of 5,000 psi (35 MPa) in 24 hours. Such concrete was produced with a variety of aggregates, including crushed granite, marine marl, dense crushed limestone, and washed rounded gravel.

2. By using insulation to trap the heat of hydration, the strength development of HES concrete can be accelerated to achieve a very early strength (VES) concrete with a strength of 2,000 psi (14 MPa) or more in 6 hours.

3. Because of a larger amount of Type III cement used in the HES concrete mixture along with a fast-acting accelerator and a relatively low water/cement ratio (W/C), the strength development of the concrete is much more rapid in the first 15 days than predicted by the current ACI Committee 318 recommendation (1993b) based on conventional concrete.

4. Because the HES concrete in this study was kept moist only for the first 24 hours followed by air curing in the laboratory, the strength development of the small laboratory samples is very rapid during the first day and the subsequent rate of strength growth is greatly reduced. The same is true for the modulus of elasticity.

5. Since the design strength (i.e., 5,000 psi or 35 MPa) of HES concrete is within the range of conventional concrete, the mechanical behavior of HES concrete, such as the modulus of elasticity and the compressive and tensile strain capacities, is similar to that of conventional concrete. The modulus of elasticity, the flexural modulus, and the splitting tensile strength can all be predicted quite well by the ACI Code equations

(1993b). The compressive strain capacity ranges from 1,500 to 2,000 microstrains, and the tensile strain capacity ranges from 150 to 250 microstrains.

6. The stress-strain relationship of HES concrete is more nonlinear at 1 day than at later ages, and the modulus of elasticity is lower for concrete with softer aggregate (e.g., marine marl) or with aggregate that contains more moisture (e.g., washed rounded gravel).

7. At the design age of 1 day, both the strength and the elastic modulus of the HES concrete with latex are somewhat lower than those of the comparable concrete without latex due to a short curing time. However, the concrete with latex, which showed a compressive strain capacity of nearly 3,000 microstrains at maximum strength, is far more ductile than the concrete without latex, which showed a compressive strain capacity of 1,500 microstrains at maximum strength.

8. Even though its W/C is low, HES concrete should have an adequate amount of air entrainment to enhance its freeze-thaw resistance. The results of this investigation indicate that HES concrete will meet the stringent requirement of a durability factor of 80% (as compared to 60% for conventional concrete) after 300 cycles of freezing and thawing according to ASTM C 666, procedure A, if the concrete contains at least 5% entrained air.

9. Shrinkage of HES concrete follows the general trend of conventional concrete. The average shrinkage strain of HES concrete at 90 days ranges from 210 to 481 microstrains, depending on the type of coarse aggregate used. These values represent 30% to 70% of the ultimate shrinkage strain recommended by ACI Committee 209 (1993a) for conventional concrete.

10. The normal procedure of the rapid chloride permeability test (RCPT) is to measure the total electrical charge (in coulombs) flowing through a vacuum-saturated concrete specimen in 6 hours. This measurement is regarded as an indication of chloride ion permeability of the concrete. HES concrete may exhibit high chloride permeability according to the RCPT since many additional ions introduced into the concrete by the various admixtures will cause the concrete to be more electrically conductive and make it appear to be more permeable than it really is.
11. The initial current (in amperes) flowing through the concrete specimen in the RCPT correlates consistently with the total charge measured in 6 hours. Therefore the initial current, which is an indirect measure of concrete conductance, can be used as an alternate measurement for the RCPT. The total testing time can thus be shortened by 6 hours.

12. The AC impedance test measures the total resistance (in ohms) of a concrete specimen. This test method is simpler and faster than the RCPT and has the potential to be used as a substitute for the RCPT. The best correlation between the two test

methods is to express the inverse impedance (reciprocal of impedance) in terms of the initial current measured in the RCPT.

13. Concrete-to-concrete bond strength can be determined by a direct shear test. The HES concrete with crushed granite developed a nominal bond strength of 275 psi (1.93 MPa)with the normal North Carolina Department of Transportation (NCDOT) pavement concrete. The HES concrete with marine marl developed a nominal bond strength of 350 psi (2.45 MPa) with the same NCDOT pavement concrete. These values are comparable to the corresponding value of 330 psi (2.31 MPa) obtained from the control test using the NCDOT concrete.

14. The concrete-to-steel bond tests indicated that the HES concrete with crushed granite developed sufficient bond strength with steel to satisfy the ACI 318 requirement on development length.

15. The experience gained from the five field experiments indicates that the slump and air content of HES concrete are much more difficult to control in the field than in the laboratory. This is not unlike the case with conventional concrete.

16. At the various ages of the HES concrete, the strengths of the cores taken from the experimental pavements generally correlate well with the strengths of the control cylinders.

17. The results of the RCPT performed on the cores taken from the experimental pavement in North Carolina confirm the results obtained from the specimens prepared in the laboratory.

18. The temperature history of the control cylinders cured in an insulated curing box in the field generally corresponds very well with the temperature history of the pavement.

19. To produce HES concrete in the field, thorough mixing of the concrete is critical. Batch size should be limited to no more than one-half to two-thirds of the rated capacity of the ready-mix concrete truck.

20. To optimize the performance of any field installation, preconstruction meetings should be held with contractors, concrete suppliers (including batch plant operators), and appropriate personnel of highway agencies.

21. Laboratory and field trial batches should be produced in sufficient numbers to confirm the mixture proportions, batching sequence, and workability of the concrete.

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Appendix A

List of Figures

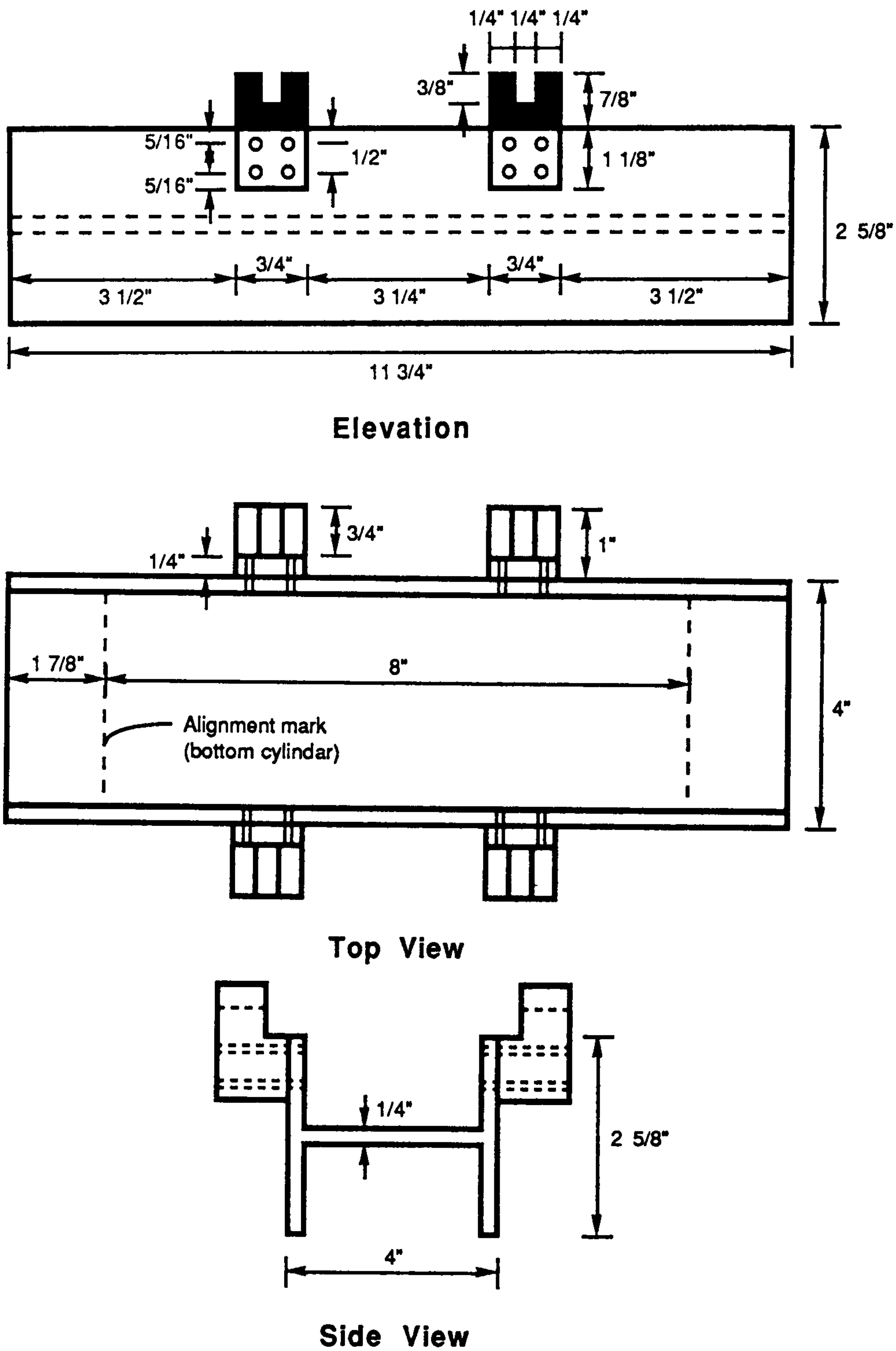
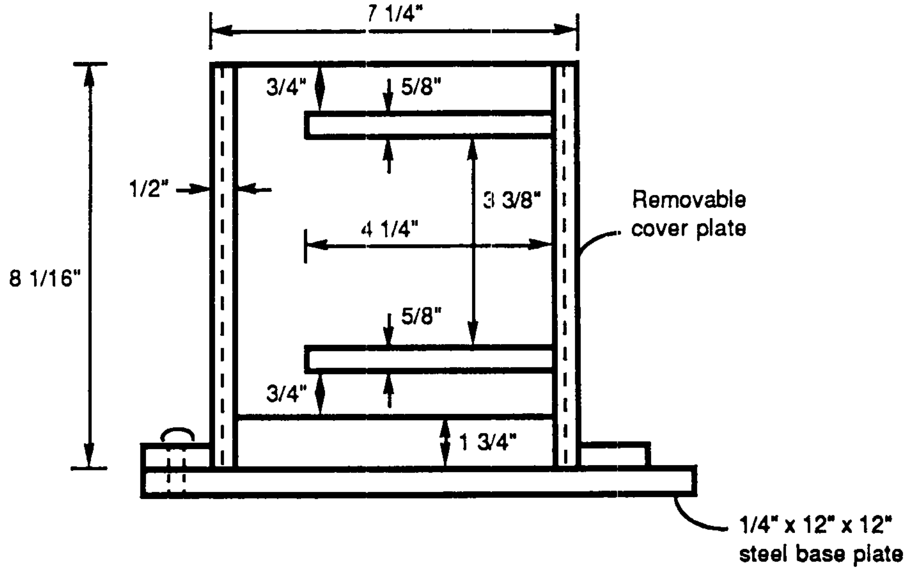
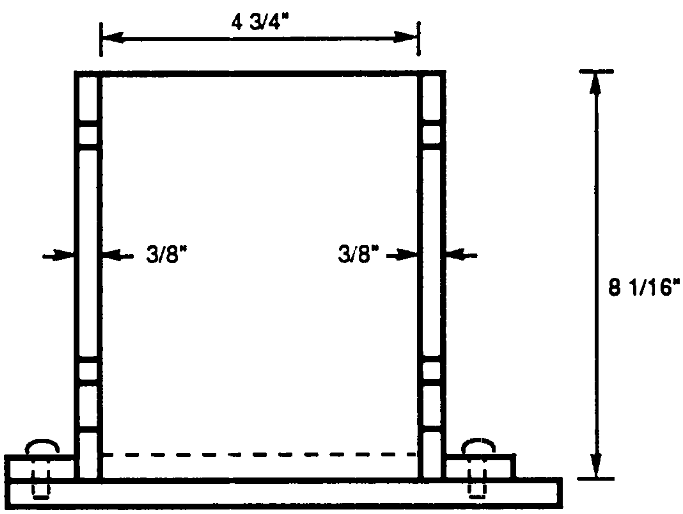


Figure A.1 Device for mounting transducers on 4 x 8-in. cylinders



Side View



Front View (without cover plate)

Figure A.2 **Steel jackets for protecting transducers during compression testing**

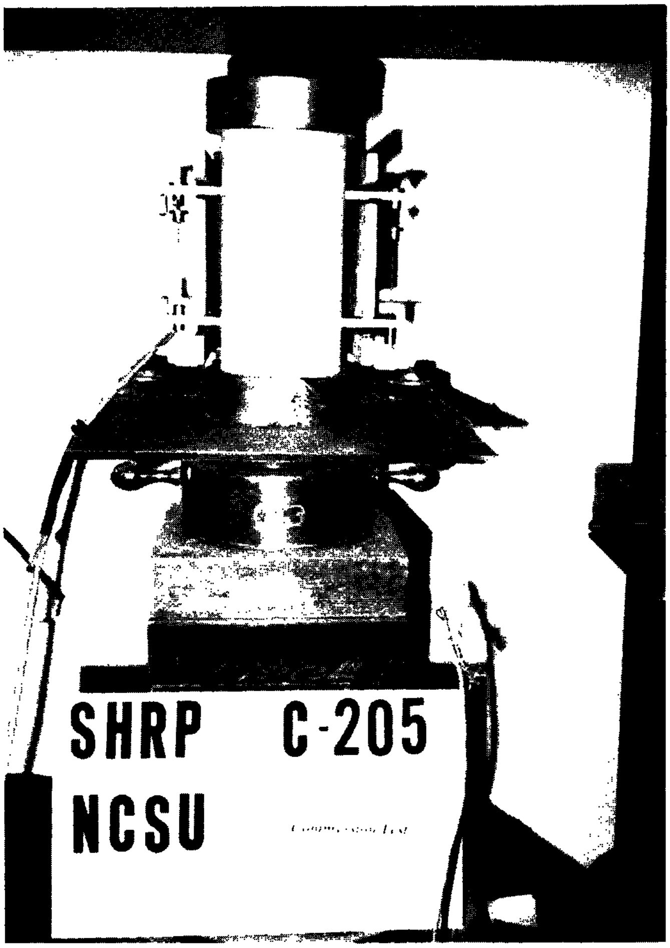


Figure A.3 Compression test setup

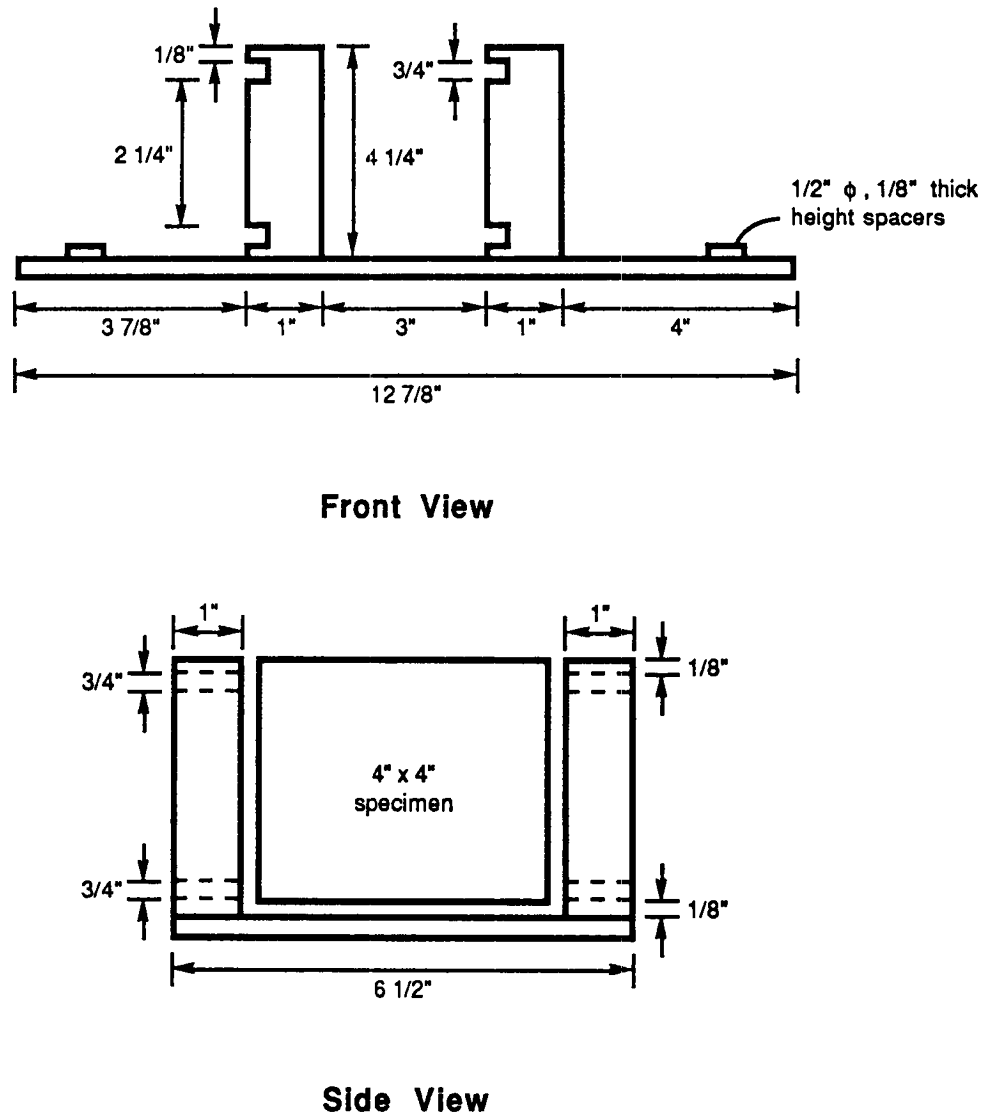
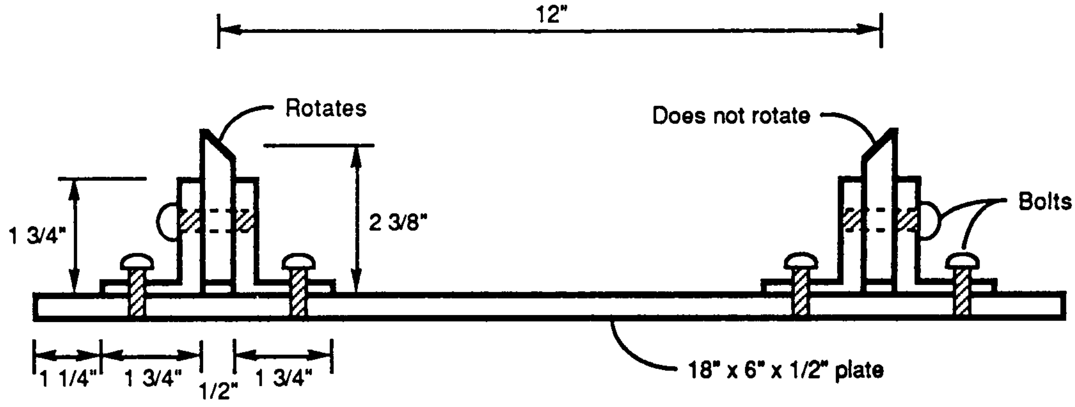
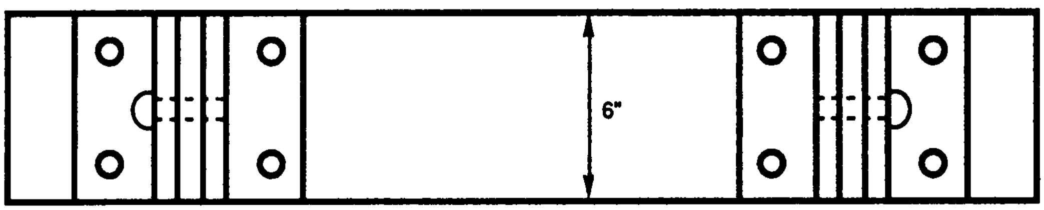


Figure A.4 Transducer mounting device for flexural testing

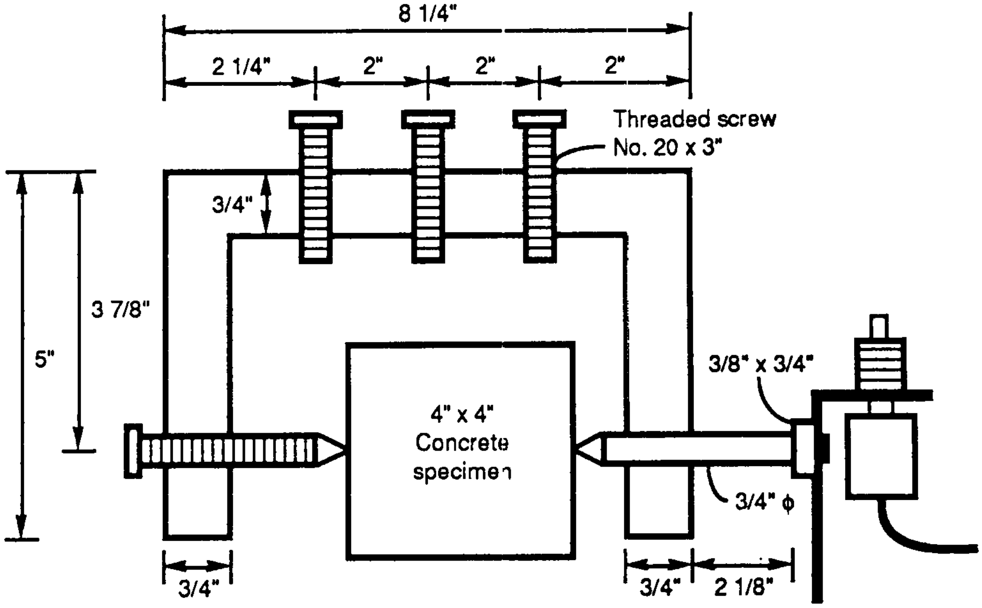


Side View

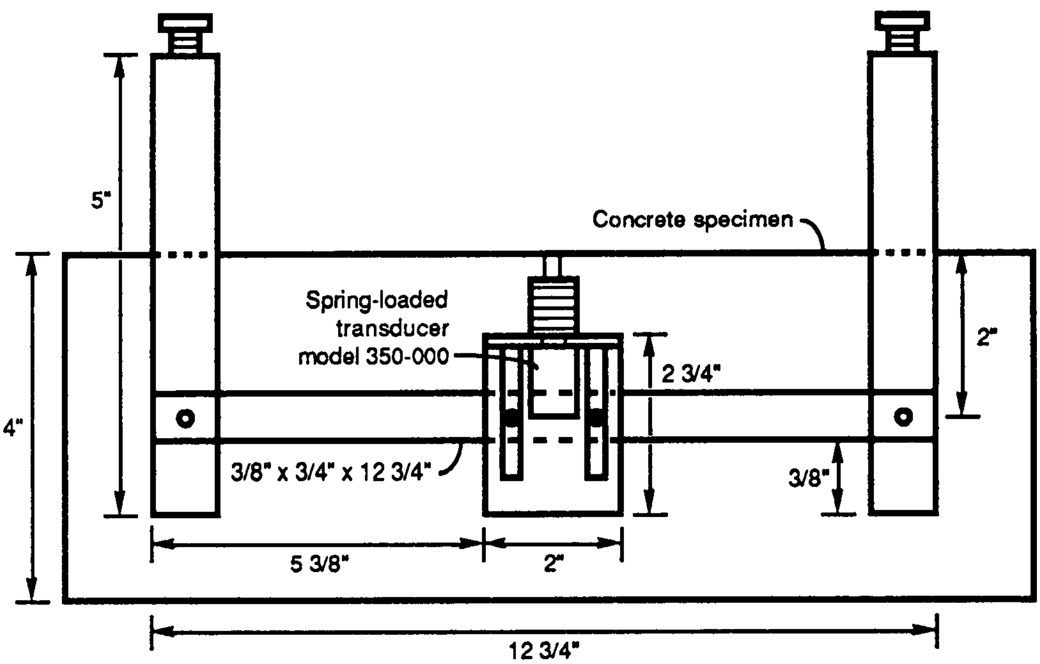


Plan View

Figure A.5 Beam support for flexural testing



Side View



Front View

Figure A.6 Frame mounting on flexural test beams for recording mid-span deflection

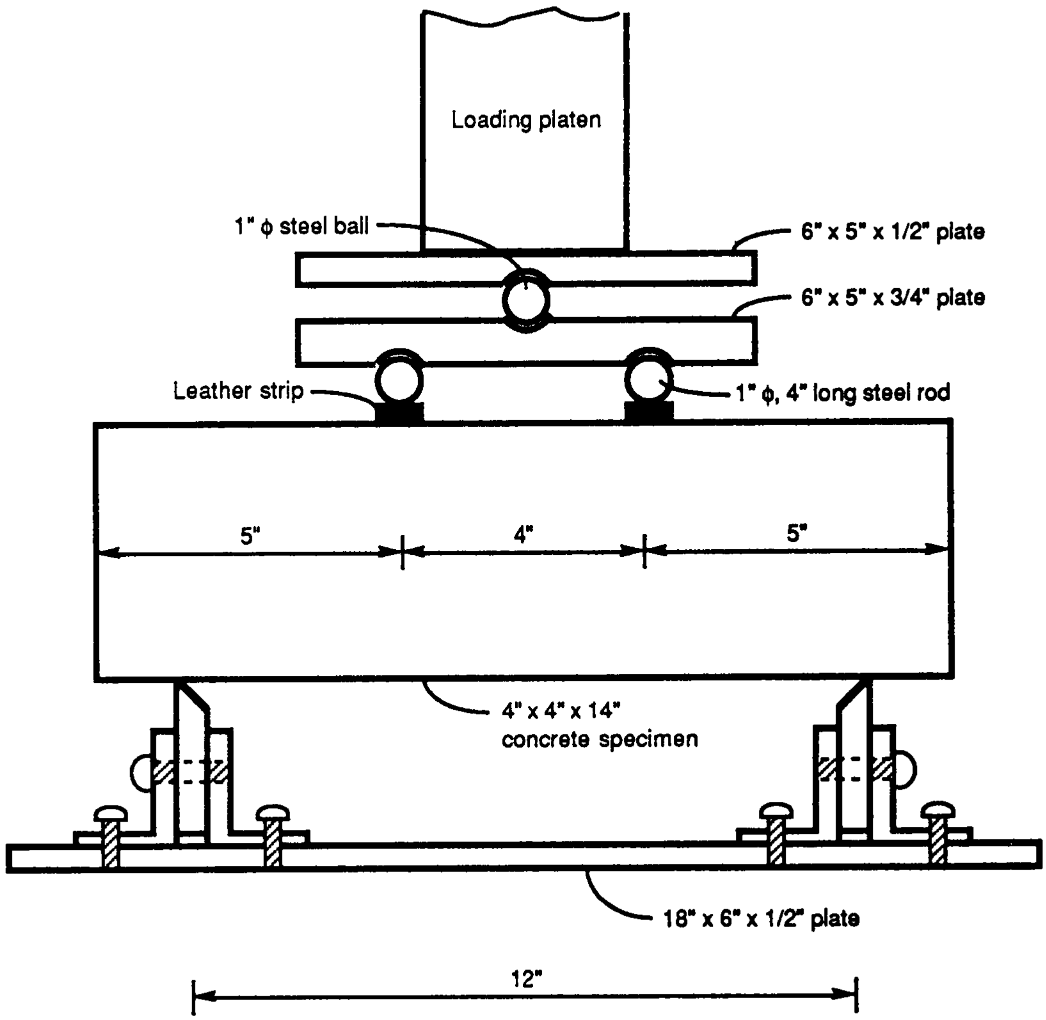


Figure A.7 Loading arrangement for flexural test

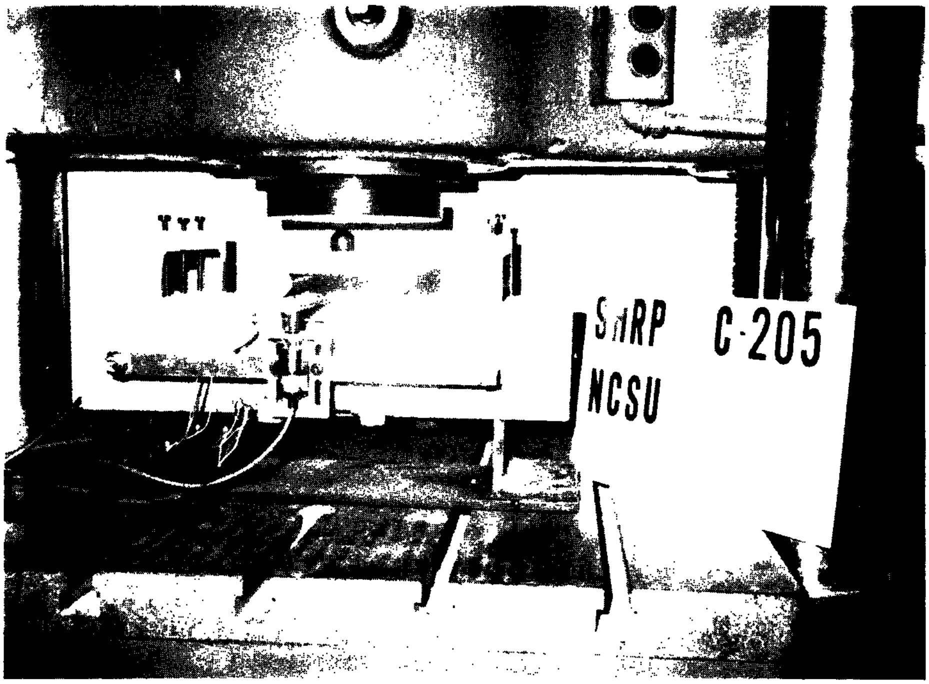


Figure A.8 Flexural test setup

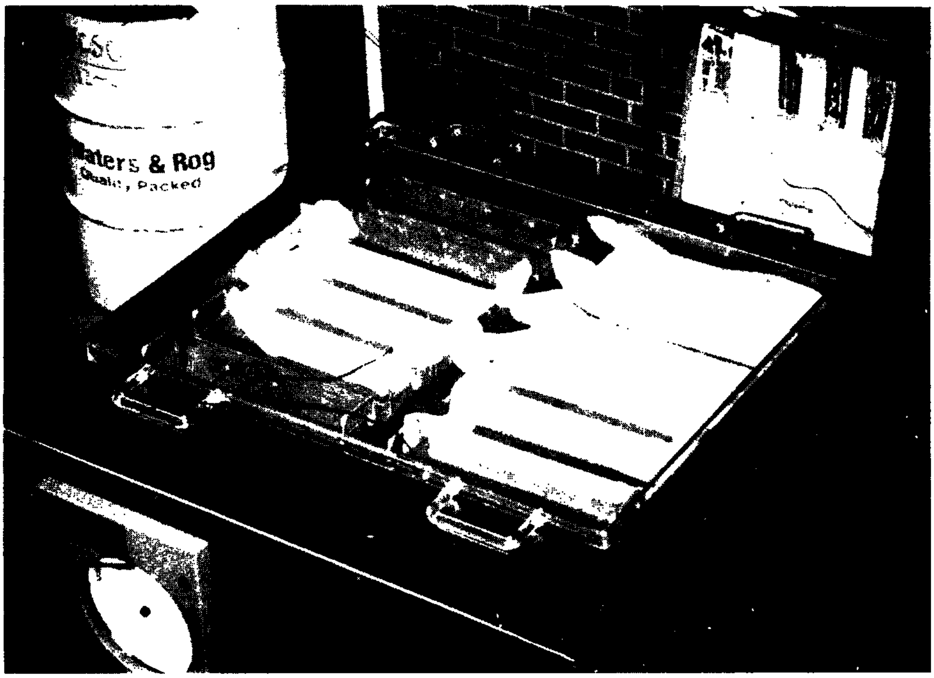


Figure A.9 Freezing-thawing chamber

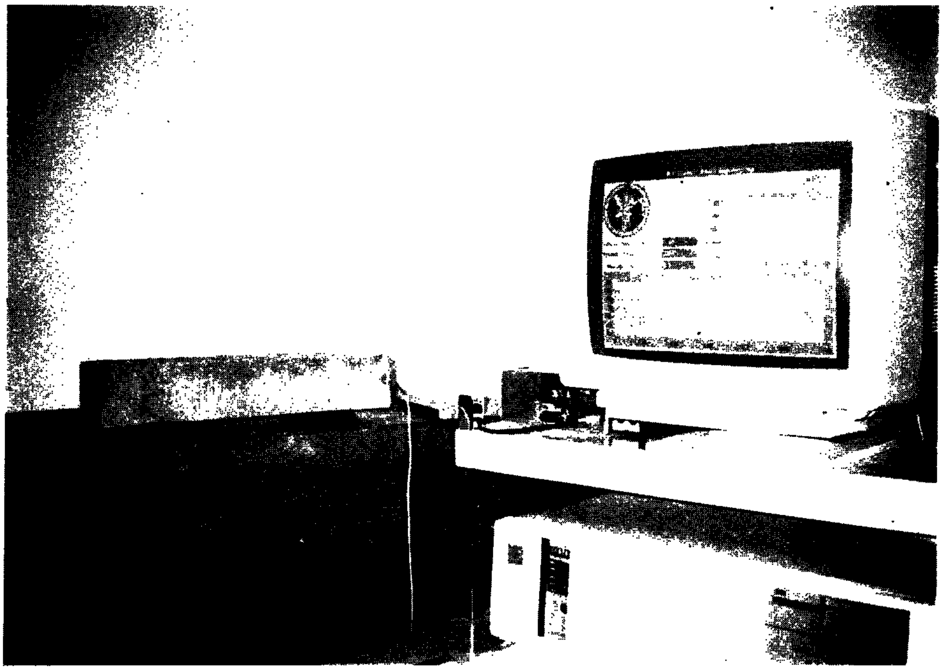


Figure A.10 Measurement of dynamic modulus of elasticity

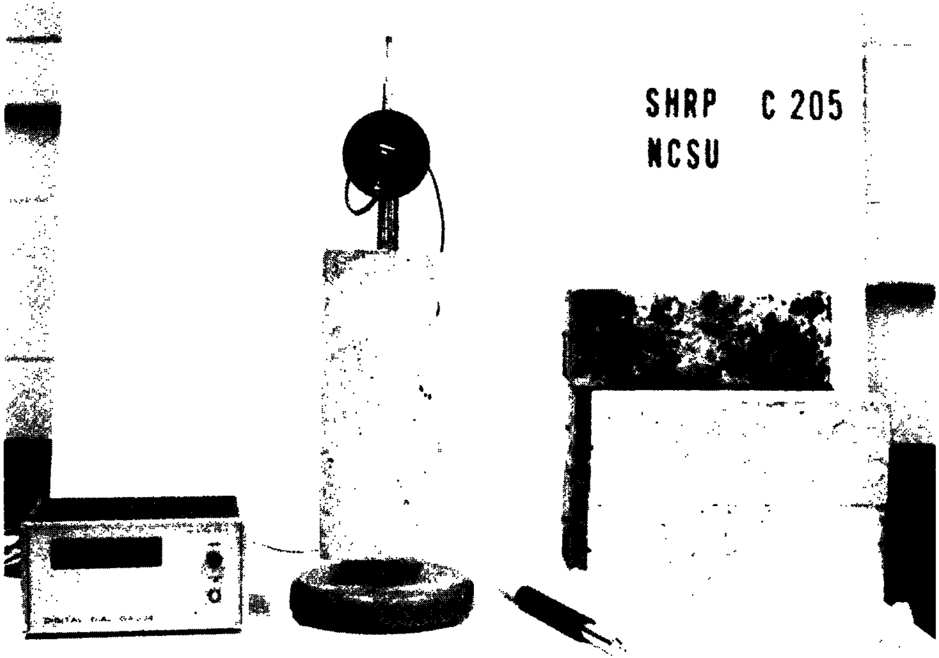


Figure A.11 Shrinkage test setup

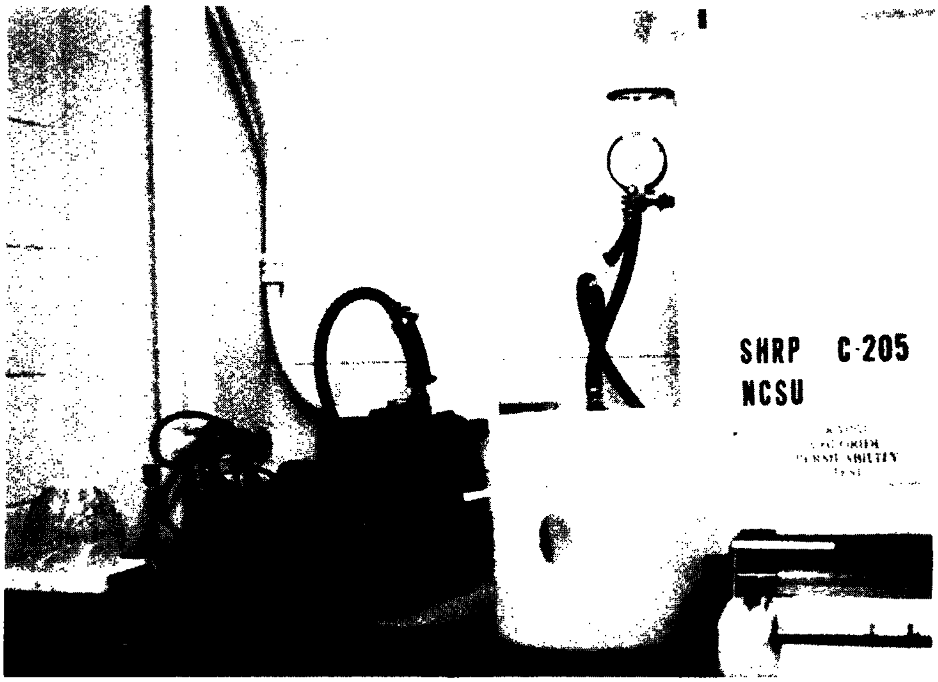


Figure A.12 Rapid chloride permeability test (RCPT) vacuum saturation setup

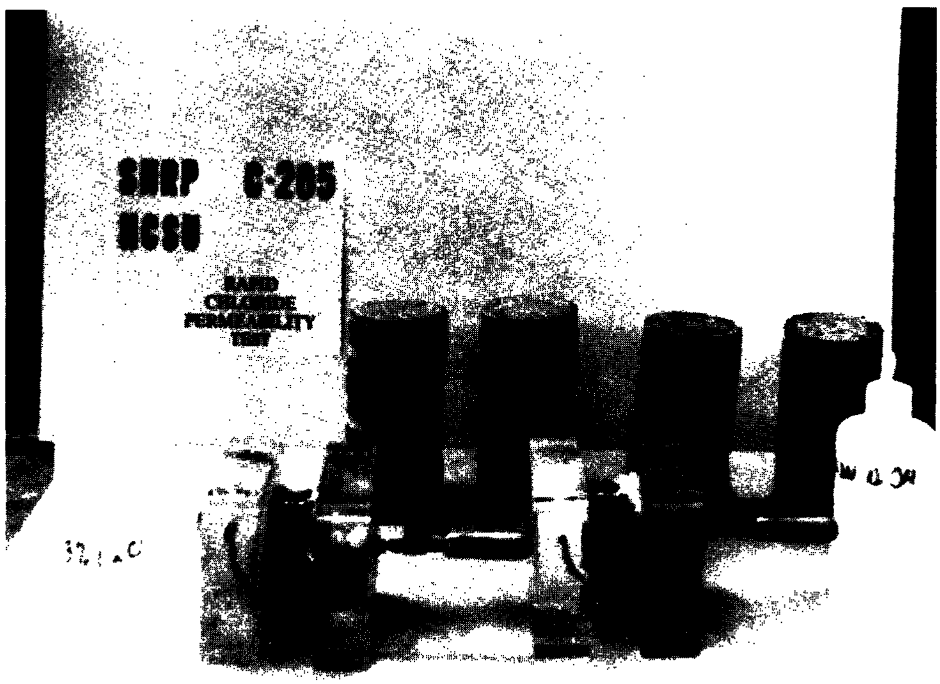


Figure A.13 RCPT specimens sealed in two test cells

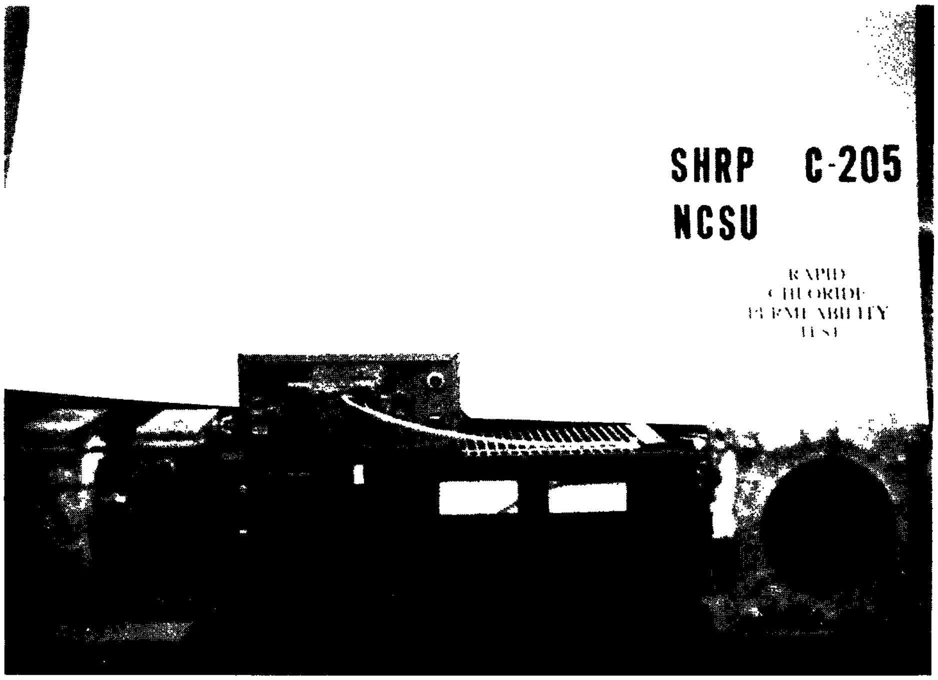


Figure A.14 RCPT test cells attached to power supply

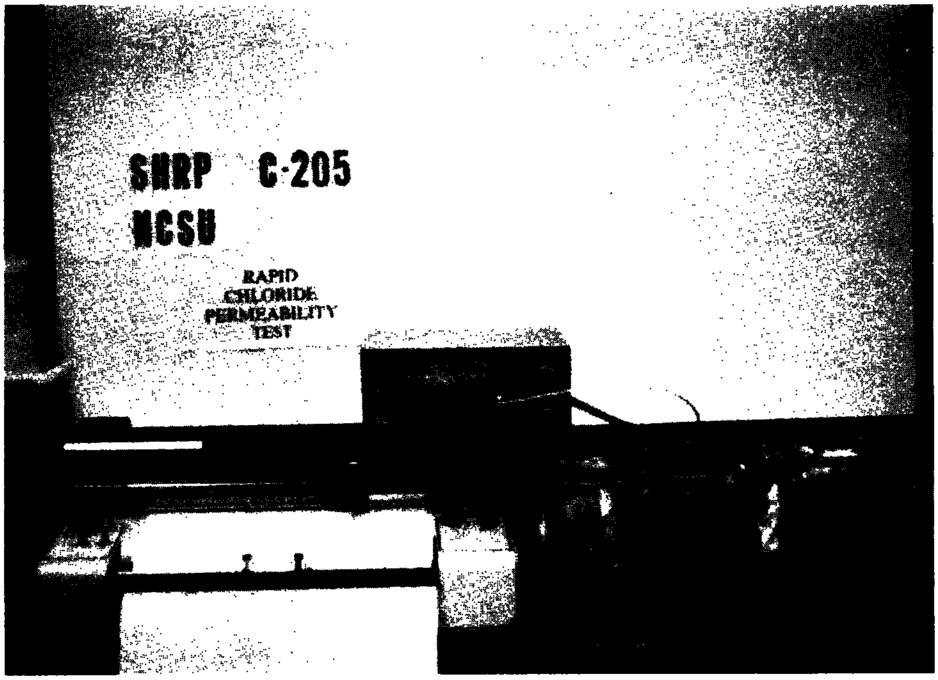


Figure A.15 RCPT output recording

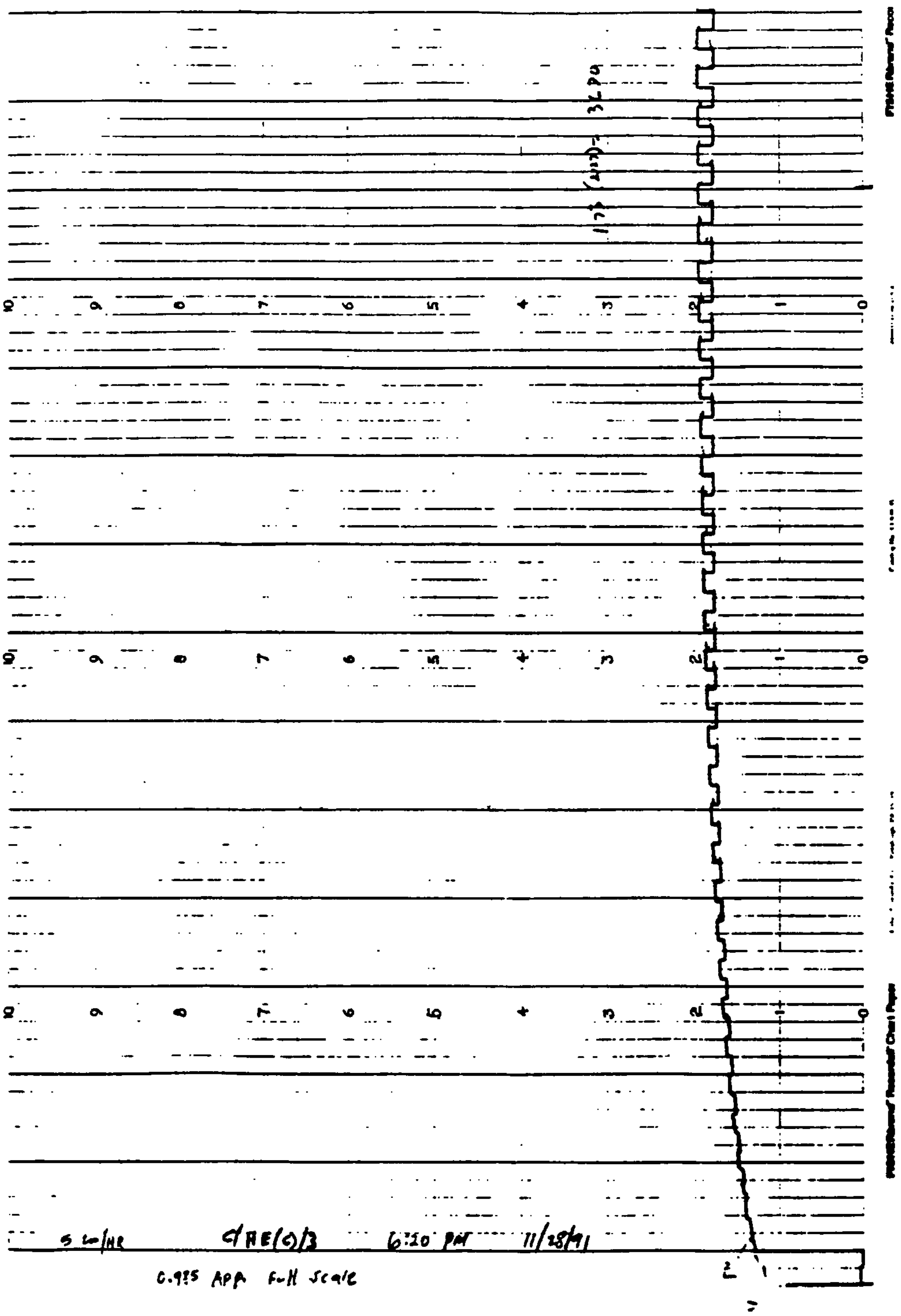


Figure A.16 RCPT output strip chart

Figure A.17 AC impedance test setup

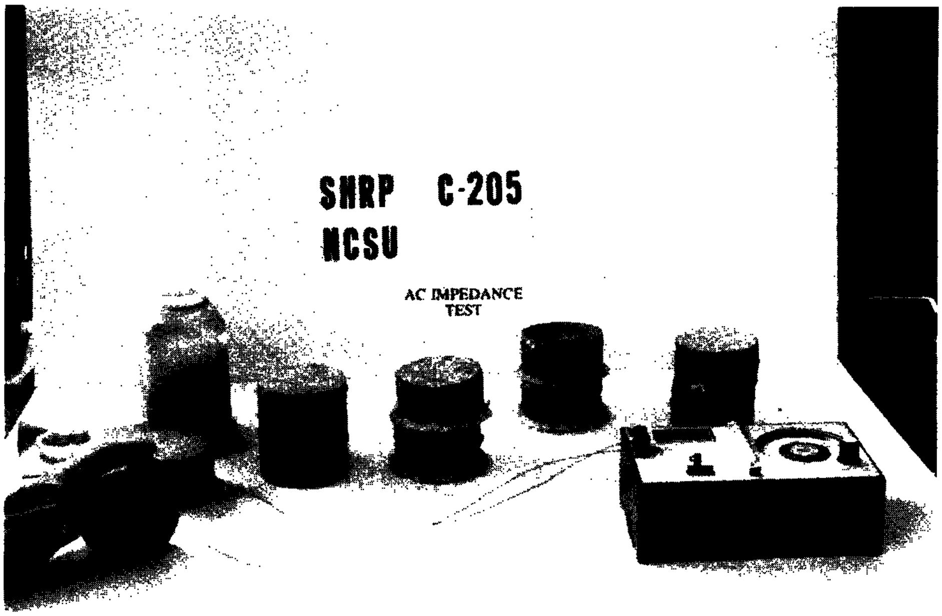


Figure A.18 AC impedance test in progress

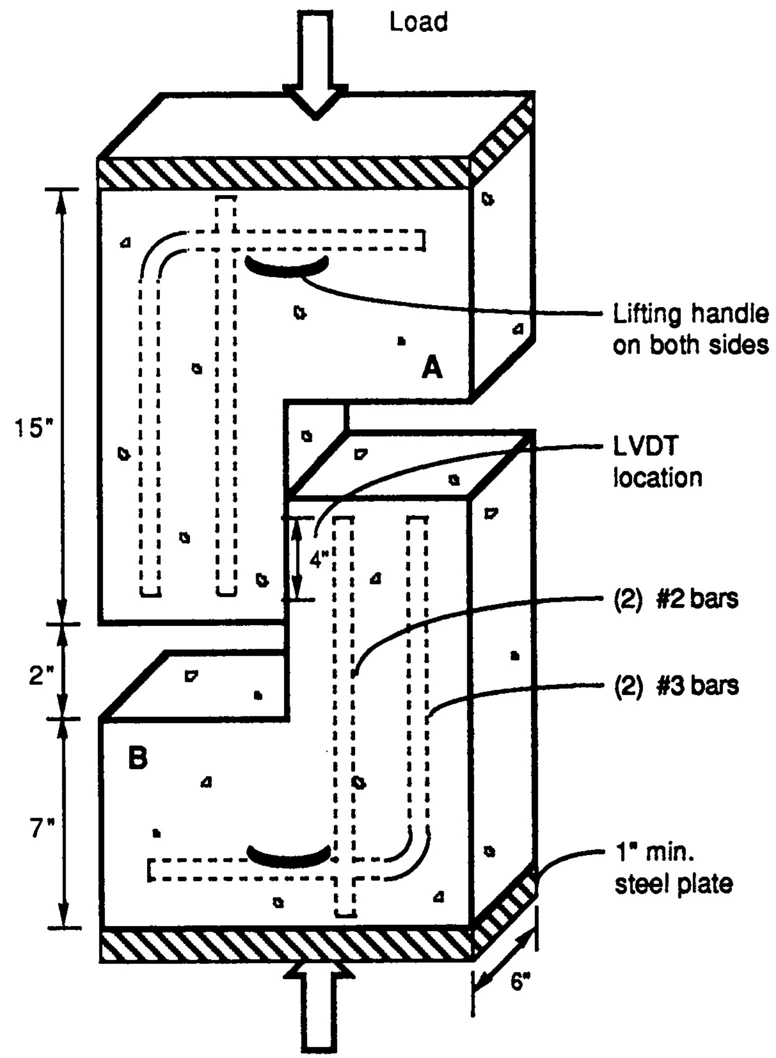


Figure A.19 Loading arrangement for concrete-to-concrete bond test

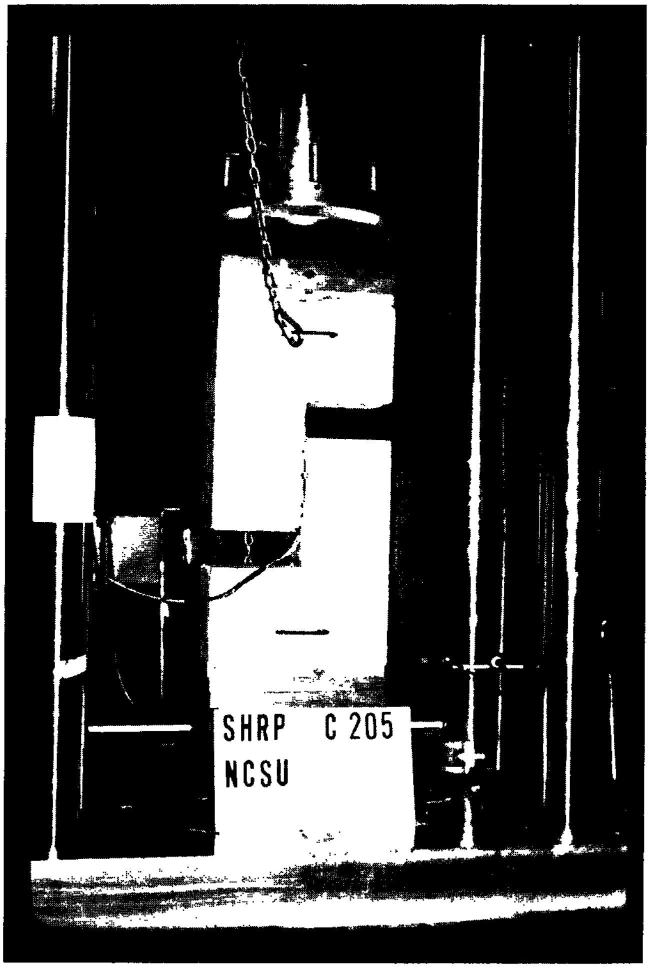


Figure A.20 Concrete-to-concrete bond test setup

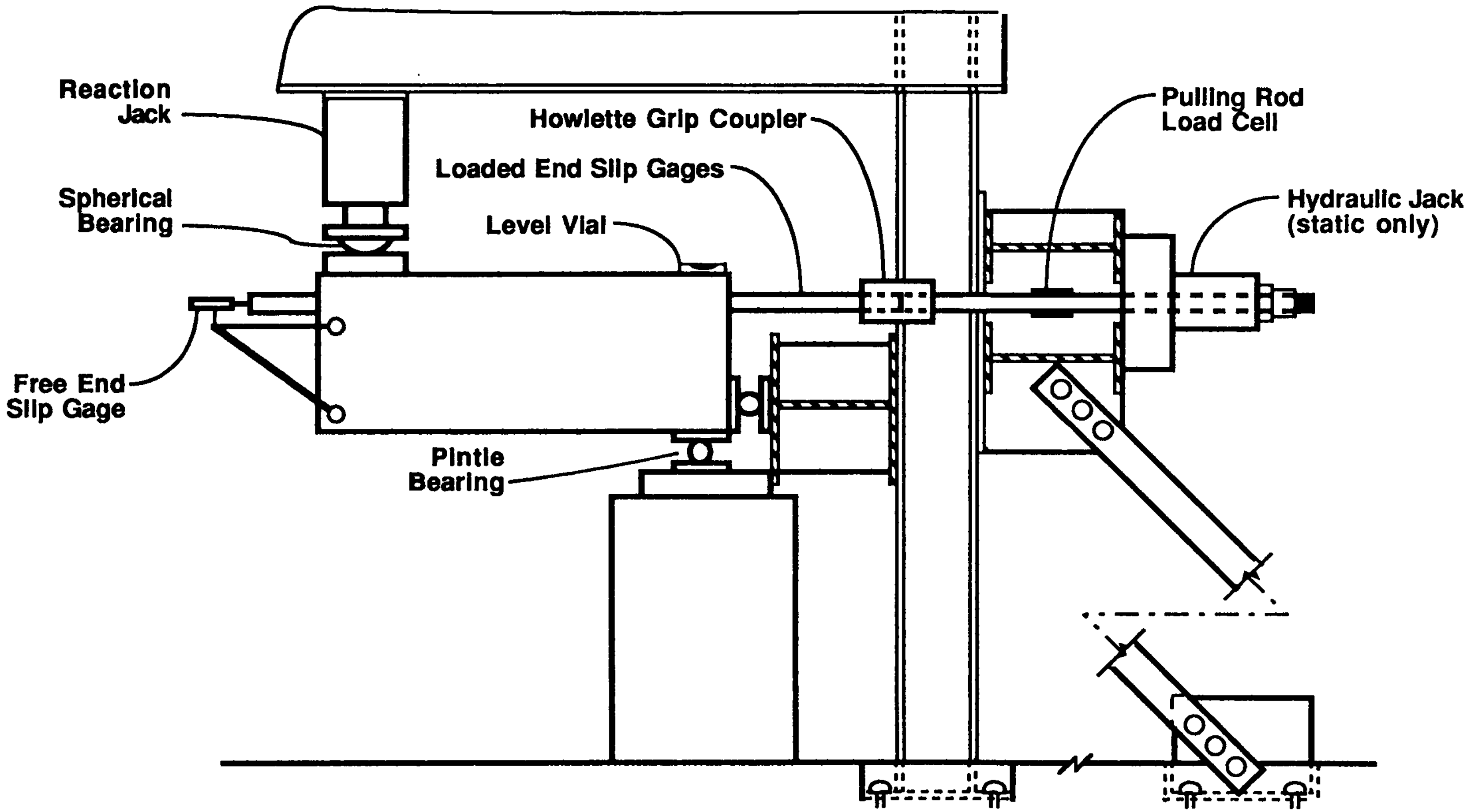


Figure A.21 **Loading arrangement for concrete-to-steel bond test**

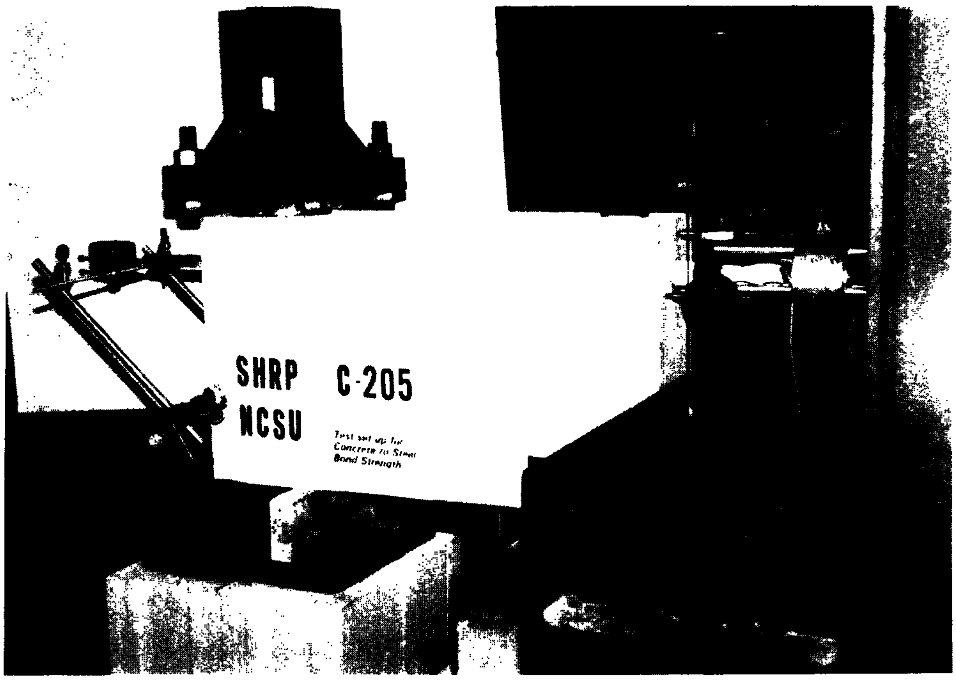


Figure A.22 Concrete-to-steel bond test setup

Appendix B

List of Tables

Table B.1 HES mix proportions per cubic yard proposed to NYDOT

Material	Mix 1	Mix 2
Cement (Type III) (lb)	780	810
Water* (lb)	270	275
Coarse aggregate (lb)	1700	1700
Fine aggregate (lb)	1000	1000
HRWR (napthalene-based) (oz/cwt)	12	14
AEA (vinsol resin) (oz/cwt)	—	1.8
Calcium nitrite (gal)	5.4	6.0

*Adjusted for free aggregate moisture and water in the calcium nitrite.
1 lb/yd^3 = 0.5933 kg/m^3, 1 gal = 3.78 L, 1 oz/cwt = 0.652 mL/kg

Table B.2 Trial batch proportions per cubic yard — NYDOT

Material	Control	Batch 1	Batch 2	Batch 3	Batch 4	Batch 5
Cement (Type III) (lb)	780	780	780	810	810	810
Water* (lb)		270	270	270	275	275
Coarse aggregate (lb)	1865	1865	1865	1803	1803	1803
Fine aggregate (lb)	995	995	995	1072	1072	1072
HRWR (WRDA-19) (oz)	93.6	117	117	153.9	186.3	170.1
AEA (Daravair) (oz)	15.6	31.2	39	48.6	56.7	48.6
Calcium nitrite (DCI) (gal)	5.4	5.4	6	6	6	6.0

* Adjusted for free aggregate moisture and water in the calcium nitrite.
1 lb/yd^3 = 0.5933 kg/m^3, 1 gal = 3.78 L

Table B.3 Trial batch properties — NYDOT

Property	Control	Batch 1	Batch 2	Batch 3	Batch 4	Batch 5
Slump (in.)	2.25	1.25	1.00	1.75	3.50	2.00
Air content (%)	3.8	3.7	3.8	5.0	10.0	6.4
Unit weight (lb/ft^3)	148.67	148.9	148.02	146.41	137.54	143.87
Compressive Strength (psi)						
4 Hours	—	130	210	500	110	150
5 Hours	725	720	850	710	*1,030	1,090
6 Hours	2,620	—	—	2,340	1,280	1,870
24 Hours	4,750	5,490	5,870	5,360	4,710	5,190
7 Days	5,620	7,160	6,860	6,970	6,020	6,660

*5.5 hour break
1 in. = 25 mm, 1 lb/ft^3 = 16 kg/m^3, 145 psi = 1 MPa

Table B.4 Aggregate gradation and material properties — ILDOT

Sieve Size	Percent Passing: Coarse	Percent Passing: Fine
1 in.	*	
3/4 in.	*	
1/2 in.	*	
3/8 in.	*	
No. 4		99.8
No. 8		90.2
No. 16		66.9
No. 30		45.2
No. 50		15.3
No. 100		3.2
No. 200		1.0
Specific gravity	2.69	2.58
Absorption (%)	1.30	2.10

*Information not furnished by ILDOT. 1 in. = 25 mm

Table B.5 Trial bach proportions — ILDOT

Material	Quantity Per Cubic Yard
Cement (Type III) (lb)	870
Water* (lb)	311
Coarse aggregate (lb)	1,729
Fine aggregate (lb)	894
HRWR (WRDA-19) (oz/cwt)	31
AEA (oz/cwt)	4
Calcium nitrite (DCI) (gal)	4

*Adjust for free aggregate mositure and water in calcium nitrite.

Table B.6 Trial batch properties — ILDOT

Property	Value	
Slump (in.)	2.0	
Air content (%)	8.2	
Compressive strength (psi)	Insulated	Noninsulated
5 Hours	1,000	735
6 Hours	2,500	1,850
7 Hours	3,540	2,550
12 Hours	5,420	4,410
24 Hours	6,970	6,240

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7/22/93