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MONITORING BOTH SHORT- AND LONG TERM MECHANICAL PROPERTIES FOR LARGE STONE MIXTURES

Neeraj Buch, Ph.D.

PRCE

Michigan State University
Pavement Research Center of Excellence

Final Report
October, 1998

TESTING AND RESEARCH SECTION
CONSTRUCTION AND TECHNOLOGY DIVISION
RESEARCH REPORT NO. RC-1367

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Technical Report Documentation Page

1. Report No. Research Report RC-1367	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Monitoring Both Short- and Long Term Mechanical Properties for Large Stone Concrete Mixtures		5. Report Date October 1998
7. Author(s) Neeraj Buch		6. Performing Organization Code
9. Performing Organization Name and Address Michigan State University Department of Civil and Environmental Engineering East Lansing, MI 48824-1226		8. Performing Org Report No. RC-1367
12. Sponsoring Agency Name and Address Michigan Department of Transportation Construction and Technology Division P.O. Box 30049 Lansing, MI 48909		10. Work Unit No. (TRAIS) 11. Contract/Grant No.
15. Supplementary Notes		13. Type of Report & Period Covered 10/1/97-9/31/98 14. Sponsoring Agency Code

16. Abstract

This report summarizes findings of a one-year (10/1/97-9/30/98)-research study that investigated mechanical properties of large coarse aggregate concrete mixtures. The concrete samples for the laboratory investigation were obtained from: (1) I-75 inlay project from the Ohio (IM 58151-38680A) border to 12 miles north (both north and south bound lanes) constructed during August-October 1997 and (2) I-75 reconstruction project (north and south) in Wayne County between Fort Street and Grand Avenue (IM 82194-36005A) constructed during June-August 1998. The PCC mixture designs included a 40%-60% blend of 4AA/6AAA as coarse aggregates. The report details mechanical and durability properties (compressive, flexural, indirect tensile, plastic shrinkage, ...) for the sampled concrete.

Some of the key conclusions and recommendations drawn from this study are:

- The laboratory tested physical properties of the coarse aggregates were similar to those reported by the contractor. The 4AA and 6AAA gradations were in accordance with the MDOT specifications. Based on the laboratory testing for a fineness modulus (fine aggregates) of 2.80 and a maximum coarse aggregate size of 50 mm, the b/b_0 was 0.74.
- The compressive strength properties of both field molded and laboratory molded specimens were well above the contractor's target 28-day strength.
- There is considerable agreement between the mechanical properties of the extracted cores and the field and laboratory molded specimens.

A visual examination of the fractured beams after testing revealed excellent aggregate interlock across the crack faces. Utilization of a strong and angular coarse aggregate resulted in a greater percentage of fracture area, thus creating a ball and socket configuration. This configuration is considered to be a very important mechanism toward maintaining integrity for transferring a wheel load across a pavement crack.

17. Key Words Large coarse aggregate, mechanical properties, restrained and plastic shrinkage.		18. Distribution Statement No restrictions. This document is available to the public through the Michigan Department of Transportation.	
19. Security Classification (report) Unclassified	19. Security Classification (Page) Unclassified	21. No. of Pages 39	22. Price

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Executive Summary

Improvement of our nation's transportation infrastructure is a top priority of both the public and private sectors of the transportation communities. The upkeep of our pavement network ranks high among our premier goals because it has an impact on national productivity, national economy and international competitiveness. Among the most pressing, costly, and challenging of transportation infrastructure issues is the improvement and assurance of long-term performance of our pavement network.. One technological improvement that is showing promise as a means to assure adequate mechanical properties and durability characteristics is High Performance Concrete (HPC). Materials classified as high performance concrete construction products and components have been available and used in the United States for decades, but in recent years, our need for infrastructure improvement has accelerated HPC research and implementation for pavements and bridges. HPC characteristics such as improved strength, freeze-thaw durability, reduced permeability, etc., that are key characteristics of HPC that when incorporated into the pavement structure will result in reduced life cycle costs, user delays, and will assure long term performance with minimal rehabilitation cycles.

This report summarizes findings of a one-year (10/1/97-9/30/98)-research study that investigated mechanical properties of large coarse aggregate concrete mixtures. The concrete samples for the laboratory investigation were obtained from: (1) I-75 inlay project from the Ohio (IM 58151-38680A) border to 12 miles north (both north and south bound lanes) constructed during August-October 1997 and (2) I-75 reconstruction project (north and south) in Wayne County between Fort Street and Grand Avenue (IM 82194-36005A) constructed during June-August 1998. The PCC mixture designs included a

40%-60% blend of 4AA/6AAA as coarse aggregates. The report details mechanical and durability properties (compressive, flexural, indirect tensile, plastic shrinkage, ...) for the sampled concrete.

Some of the key conclusions and recommendations drawn from this study are:

- The laboratory tested physical properties of the coarse aggregates were similar to those reported by the contractor. The 4AA and 6AAA gradations were in accordance with the MDOT specifications. Based on the laboratory testing for a fineness modulus (fine aggregates) of 2.80 and a maximum coarse aggregate size of 50 mm, the b/b_o was 0.74.
- The compressive strength properties of both field molded and laboratory molded specimens were well above the contractor's target 28-day strength.
- There is considerable agreement between the mechanical properties of the extracted cores and the field and laboratory molded specimens.
- A visual examination of the fractured beams after testing revealed excellent aggregate interlock across the crack faces. Utilization of a strong and angular coarse aggregate resulted in a greater percentage of fracture area, thus creating a ball and socket configuration. This configuration is considered to be a very important mechanism toward maintaining integrity for transferring a wheel load across a pavement crack.

Furthermore, some recommendations for future research are:

- It is strongly recommended that an on-going (on a yearly basis) monitoring program be initiated to monitor both the mechanical and durability properties of the concrete mixture.

- For the I-75 Monroe project a comparison can be made between the large coarse aggregate mixtures used in the right truck lane and the traditional full-depth concrete mixtures, since extensive full depth patches have been placed in the middle and median left lanes.
- In order to test the premise that a reduction in cement content and addition of pozzolanic improves the volumetric properties (shrinkage and creep) of the mixture extensive plastic, and drying shrinkage tests need to be performed.
- A Volumetric Surface Texture Analysis needs to be performed on extracted PCC cores from large coarse aggregate mixture pavements to determine the distribution of macro- and micro-texture and whether this can be related to load transfer efficiency across a crack.

Chapter 1

Introduction

Improvement of our nation's transportation infrastructure is a top priority of both the public and private sectors of the transportation communities. The upkeep of our pavement network ranks high among our premier goals because it has an impact on national productivity, national economy and international competitiveness. Among the most pressing, costly, and challenging of transportation infrastructure issues is the improvement and assurance of long-term performance of our pavement network.

We need stronger and more durable portland cement concrete in the rigid pavement network to resist (1) the ever increasing truck weights, tire pressures, traffic volumes and (2) the environmental effects on fresh and hardened concrete properties. One technological improvement that is showing promise as a means to assure adequate mechanical properties and durability characteristics is High Performance Concrete (HPC). Materials classified as high performance concrete construction products and components have been available and used in the United States for decades, but in recent years, our need for infrastructure improvement has accelerated HPC research and implementation for pavements and bridges. HPC characteristics such as improved strength, freeze-thaw durability, reduced permeability, etc., that are key characteristics of HPC that when incorporated into the pavement structure will result in reduced life cycle costs, user delays, and will assure long term performance with minimal rehabilitation cycles. Table 1 summarizes the performance characteristics for high performance structural concrete.

Table 1: Grades of performance characteristics for high performance structural concrete (1).

GRADES OF PERFORMANCE CHARACTERISTICS FOR HIGH PERFORMANCE STRUCTURAL CONCRETE[1]

PERFORMANCE	STANDARD	FHWA HPC PERFORMANCE GRADE[3]				
CHARACTERISTIC[2]	TEST METHOD	1	2	3	4	N/A
FREEZE/THAW DURABILITY[4] (χ=relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666 Proc. A	$60\% \leq \chi < 80\%$	$80\% \leq \chi$			
SCALING RESISTANCE[5] (χ=visual rating of the surface after 50 cycles)	ASTM C 672	χ=4,5	χ=2,3	χ=0,1		
ABRASION RESISTANCE[6] (χ=avg. depth of wear in mm)	ASTM C 944	$2.0 > \chi \geq 1.0$	$1.0 > \chi \geq 0.5$	$\chi < 0.50$		
CHLORIDE PERMEABILITY[7] (χ=coulombs)	AASHTO T 277 ASTM C 1202	$3000 \geq \chi > 2000$	$2000 \geq \chi > 800$	$\chi \leq 800$		
STRENGTH (χ=compressive strength)	AASHTO T 22 ASTM C39	$41 \leq \chi < 55$ MPa ($6 \leq \chi < 8$ ksi)	$55 \leq \chi < 69$ MPa ($8 \leq \chi < 10$ ksi)	$69 \leq \chi < 97$ MPa ($10 \leq \chi < 14$ ksi)	$\chi \geq 97$ MPa ($\chi \geq 14$ ksi)	
ELASTICITY[10] (χ=modulus of elasticity)	ASTM C 469	$28 \leq \chi < 41$Gpa ($4 \leq \chi < 6 \times 10^6$ psi)	$40 \leq \chi < 50$ Gpa ($6 \leq \chi < 7.5 \times 10^6$ psi)	$\chi \geq 50$ Gpa ($\chi \geq 7.5 \times 10^6$ psi)		
SHRINKAGE[8] (χ=microstrain)	ASTM C 157	$800 > \chi \geq 600$	$600 > \chi \geq 400$	$\chi < 400$		
CREEP[9] (χ=microstrain/pressure unit)	ASTM C 512	$75 \geq \chi > 60$ / MPa ($0.52 \geq \chi > 0.41$ / psi)	$60 \geq \chi > 45$ / MPa ($0.41 \geq \chi > 0.31$ / psi)	$45 \geq \chi > 30$ / MPa ($0.31 \geq \chi > 0.21$ / psi)	$\chi \leq 30$ MPa ($\chi \leq 0.21$ psi)	

[1]This table does not represent a comprehensive list of all characteristics that good concrete should exhibit. It does list characteristics that can quantifiably be divided into different performance groups. Other characteristics should be checked. For example, HPC aggregates should be tested for detrimental alkali silica reactivity according to ASTM C277, cured at 38°C and should yield less than 0.05% mean expansion at 3 months and less than 0.10% expansion at 6 months (based on SHRP C-342, p.83). Due consideration should also be paid to (but not necessarily limited to) acidic environments and sulfate attack.
[2]All tests to be performed on concrete samples moist or submersion cured for 56 days. See Table 3 for additional test information.
[3]A given high performance concrete mix design is specified by a grade for each desired performance characteristic. For example, a concrete may perform at Grade 4 in strength and elasticity, Grade 3 in shrinkage and scaling resistance, and Grade 2 in all other categories.
[4]Based on SHRP C/FR – 91 – 103, p.3.52.
[5]Based on SHRP S-360.
[6]Based on SHRP C/FR - 91 – 103.
[7]Based on PCA *Engineering Properties of Commercially Available High-Strength Concretes.*
[8]Based on SHRP C/FR – 91 – 103, p.3.25.
[9]Based on SHRP C/FR – 91 – 103, p.3.30.
[10]Based on SHRP C/FR – 91 – 103, p.3.17.

PROBLEM STATEMENT

MDOT's current standard concrete mixture designs for rigid pavements use Michigan series 6A/6AA coarse aggregate. This aggregate series consists of a particle size distribution range from 25 mm top size to the 4.75 mm. bottom size sieves. A close inspection of the gradation requirement specifies no retention requirement on the 19 mm sieve size. Therefore, in reality, the 6A/6AA aggregates are manufactured with a top size of 19.0 mm. This gradation is preferred where "hand finishing" is an issue, but does not lend itself well from an engineering standpoint, where reliance on aggregate interlock (at a crack face) is critical to assist in maintaining vertical alignment and adequate load transfer across a longitudinal and/or transverse crack. The existing concrete mixture designs require a higher cement mortar content for adequate coating of the coarse aggregate system, resulting in excessive volume changes (shrinkage) during the early life of a concrete pavement, thereby increasing matrix permeability.

Historically, MDOT's concrete mixtures have large coarse aggregates which performed well (Woodward Avenue, I-69 near Marshall and Davison Freeway), except when aggregates susceptible to D-cracking. A large and more durable coarse aggregate would assist in maintaining pavement integrity at crack locations by providing adequate vertical shear resistance from the "ball and socket" mechanism of the large stone protruding across the crack face. The use of large stone concrete mixes may reduce the mortar content and subsequently reduce shrinkage stresses which would reduce the potential of early age cracking in pavements. This is very important for pavements and bridge decks where geometric constraints induce non-uniform levels of internal stress characteristics into the concrete. The levels of stress and strain can be reduced by the proper selection and proportioning of materials to maintain an extended pavement service life. Furthermore, the reduction in the amount of cement content may result in an overall material cost reduction.

OBJECTIVES

The objectives of this project were to:

1. Monitor mechanical properties of large coarse aggregate concrete mixtures placed at the I-75 project from the Ohio (IM 58151-38680A) border to 12 miles north (both north and south bound lanes) and I-75 (north and south) in Wayne County between Fort Street and Grand Avenue (IM 82194-36005A).
2. Monitor durability properties of these mixtures for the same above-mentioned locations.

CONTENTS OF THE REPORT

This report provides an overview of the work performed over the project year, October 1, 1997 to September 30, 1998 as well as detailed laboratory and field test results. Chapter II discusses the two field projects, types of tests performed on the laboratory and field molded samples, types of tests performed on the PCC cores extracted from the pavement sites and data from these tests. Chapter III provided conclusions based on the test results and suggestions for future research.

Chapter 2

Laboratory and Field Investigation

PROJECT 1: I-75 IN MONROE COUNTY

The construction boundaries for this project extended from station 549+47 to station 19+95 in Monroe County, University region. The truck-lane (both in the north and south directions) was reconstructed as a 11" jointed plain portland cement concrete pavement resting on an existing 16" subbase (12" original sand subbase and 4" of open graded drainage course 8G with no separator). The middle and median lanes were rehabilitated using full-depth patch repair and crack retrofitting methods. The concrete mixture design for this project is summarized in Table 2.

Table 2. Concrete Mixture Design (as provided by the contractor).

Mixture Ingredients	I-75 (Ohio Border)*
Cement	267.60
Sand, Pit#40-50	862.60
Fly Ash	42.70
Coarse Agg 1 (4AA) Pit#58-94A	379.10
Coarse Agg 2 (6AAA) Pit#58-94A	574.90
Water	138.20
AEA Axim Catexol AE260	283 mL.

*All weights are in kg/m^3

The specific gravity of the coarse aggregate (dolomitic limestone) 4AA and 6AAA was 2.643 and 2.637 respectively. The coarse aggregate 4AA and 6AAA were blended in a 40-60 ratio. The specific gravity for the fine aggregate was 2.601 and the fineness modulus was 2.80. The grain size distribution is illustrated in Figures 1 through 3. The physical properties of the blend are summarized in Table 3.

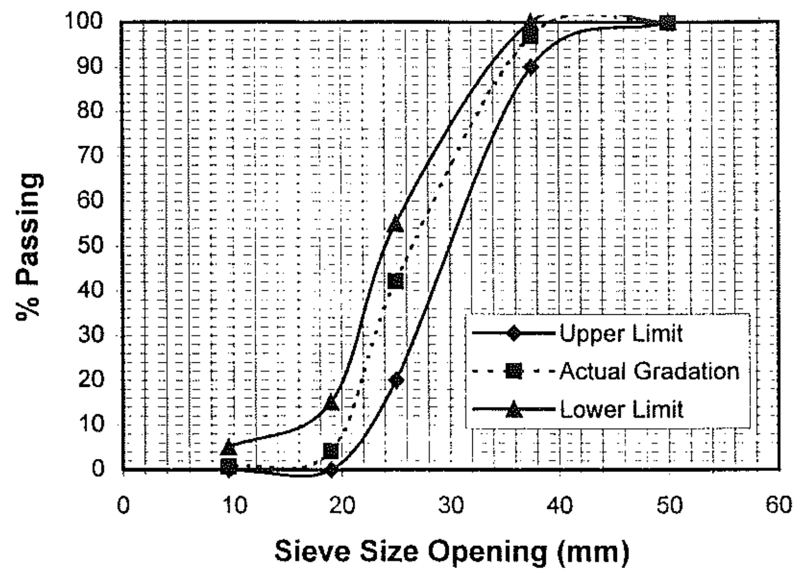

Figure 1. **Particle Size Distribution for 4AA.**

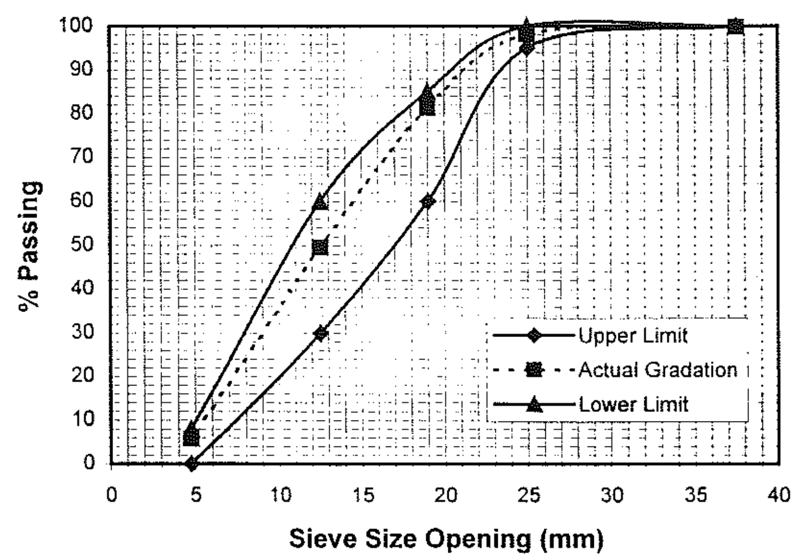

Figure 2. **Particle Size Distribution for 6AAA.**

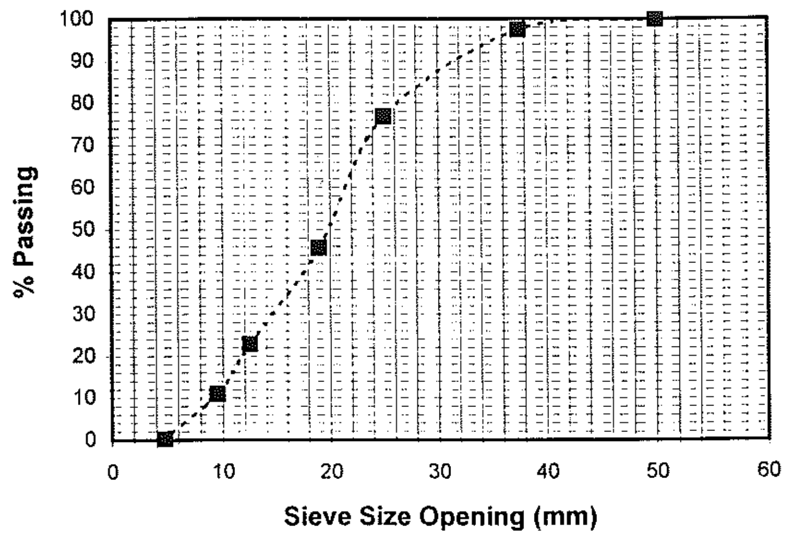

Figure 3. **Particle Size Distribution 40%(4AA)-60%(6AAA).**

Table 3. **Physical Properties of the 40-60 Blend (as determined in the laboratory).**

Property	Trial 1	Trial 2	Trial 3	Average
Sp. Gravity (bulk)	2.568	2.560	2.535	2.554
Abs. Capacity(24hr)	2.80	2.97	3.18	2.983
% Voids*	38.95	36.86	36.33	37.72
Unit Weight**	1565	1587	1611	1588

***Equation 1 was used to compute voids.**
****Dry unit weight, expressed in kg/m^3.**

$$\%\text{Voids} = \left(\frac{\text{BSG} \bullet \rho_w - \text{UW}}{\text{BSG} \bullet \rho_w} \right) \times 100 \quad \textbf{(1)}$$

where:

BSG = bulk specific gravity,
UW = unit weight aggregate, and
ρ_w = density of water.

The fresh concrete was sampled from the field for both the Northbound and Southbound lanes of Interstate 75. Three replicate samples were molded for the compressive strength, flexural strength, impact strength, and indirect tensile strength. Slump and air content tests were performed at the project site. The field mixture designs were replicated in the laboratory to mold plastic shrinkage and drying shrinkage specimens. The compressive strength test, flexural strength test, and indirect tensile strength test were performed in accordance with ASTM specifications and their procedures will not be described. The impact test, plastic shrinkage and drying shrinkage test procedures are still experimental and do not have any ASTM specification, hence the test procedures are described in the subsequent sections of this chapter.

PLASTIC SHRINKAGE TEST

Plastic shrinkage cracking is most common on horizontal pavements and slabs where rapid evaporation is possible, and its occurrence will destroy the integrity of the surface and reduce its durability. This condition is aggravated by a combination of high wind velocity, low relative humidity and high air temperature. Plastic shrinkage cracking occurs in the superficial layer of fresh concrete during the first few hours after placement. The principal cause of this type of cracking is an excessively rapid evaporation of water from the concrete surface, such that it exceeds the rate at which bleed water rises to the surface. The formation of plastic shrinkage cracking takes place when internal stress is higher than the tensile strength of concrete. The internal stress is closely related to the capillary pressure of the pore water within the fresh concrete. Once the concrete surface has attained some initial rigidity, it may not be able to accommodate plastic shrinkage by plastic flow, and thus plastic shrinkage cracks may develop. The plastic shrinkage cracking mechanism is shown in Figure 4.

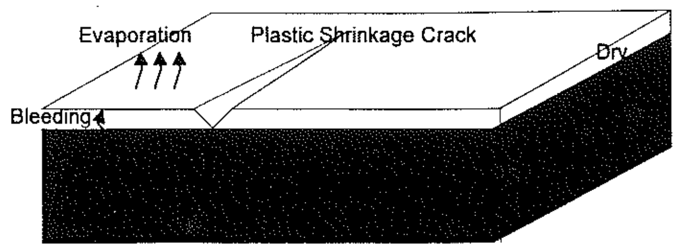

Figure 4. Plastic Shrinkage Cracking Mechanism.

The plastic shrinkage test method is under review by the ASTM C.09.03.04 Task Group on Shrinkage Testing Draft 6 June 1992 (2). This test method compares the surface cracking of a large coarse aggregate mixture panel with the surface cracking of control mix panel. Values of shrinkage cracking stated in mm^2 are regarded as the standard.

The specimens were molded using a plyform mold with a surface area to volume ratio of 3:1. Specimen dimensions were 356 mm in width x 597 mm in length x 100 mm in depth. A central metal riser was inserted in the bottom of the mold and oiled to allow for debonding of the plastic concrete. This serves as the stress riser. Risers at each end of the mold provide end restraint. A schematic of the test set-up is shown in Figure 5. Specimens were vibrated for a total of 15 seconds. The shrinkage specimens were then moved to the wind tunnel where they were subjected to dry and hot windy conditions. These conditions were produced by passing hot air at approximately 100^0F with a relative humidity between 10 and 20 percent at a speed of 9.0 to 10.0 m/sec. Fans and heaters were turned on, pushing dry hot air across the specimens. Wind testing was conducted for a total of three hours after which the specimens were removed from the tunnel and the length and width of the cracks were measured. The width of each crack was measured with a crack comparator at approximately every inch along its length. Crack lengths and widths were then multiplied to compute the area of one crack. The procedure was repeated for each crack and the total crack area was calculated by summing up individual values. The total crack is referred to as cracking value. For large stone mixes, cracking ratio relative to control mix is determined as :

$$\text{Cracking Ratio} = \frac{\text{Cracking Value for large stone mixture}}{\text{Cracking Value for control mixture}} \tag{2}$$

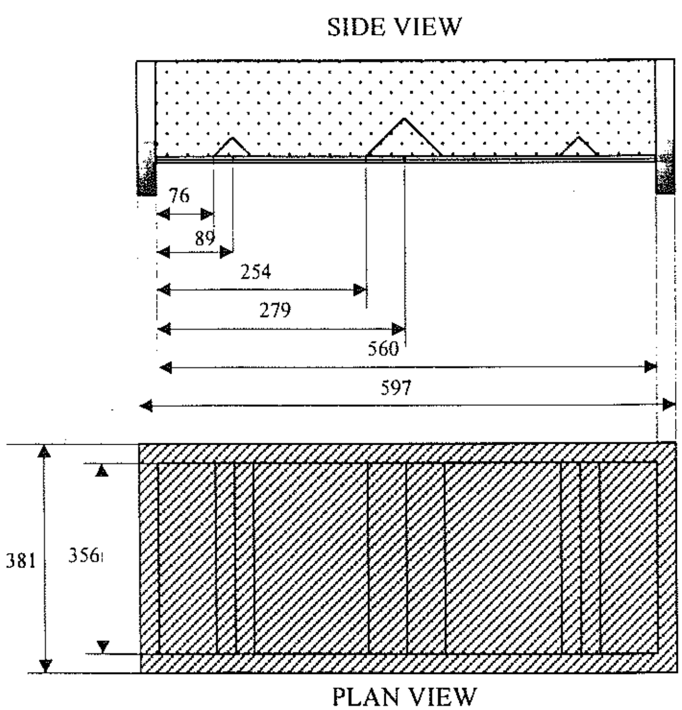

Figure 5 Specimen geometry and configuration (2).

IMPACT RESISTANCE

All concrete is brittle in nature, and plain concrete is especially suspect because it has no reinforcement to bridge the cracks. This is one of the biggest drawbacks of using plain concrete in pavements, which are subjected to various types of impact loads. The impact resistance of concrete can be measured by the impact resistance test as recommended by ACI Committee 544.2R-89 (3). This test is a "repeated impact" drop-weight test. This test gives the number of blows necessary to cause prescribed level of distress in the concrete in the test specimen. The equipment consists of :

- A standard manually operated 4.46 Kg (10 lbs) compaction hammer with a drop of 457 mm (18-in.),
- 63.5 mm (2 ½ in.) diameter steel ball,
- a flat base plate with positioning fixtures as shown in Figure 6.

The hammer is dropped repeatedly, and the number of blows required to cause the first visible crack on the top as well as to cause ultimate failure are recorded.

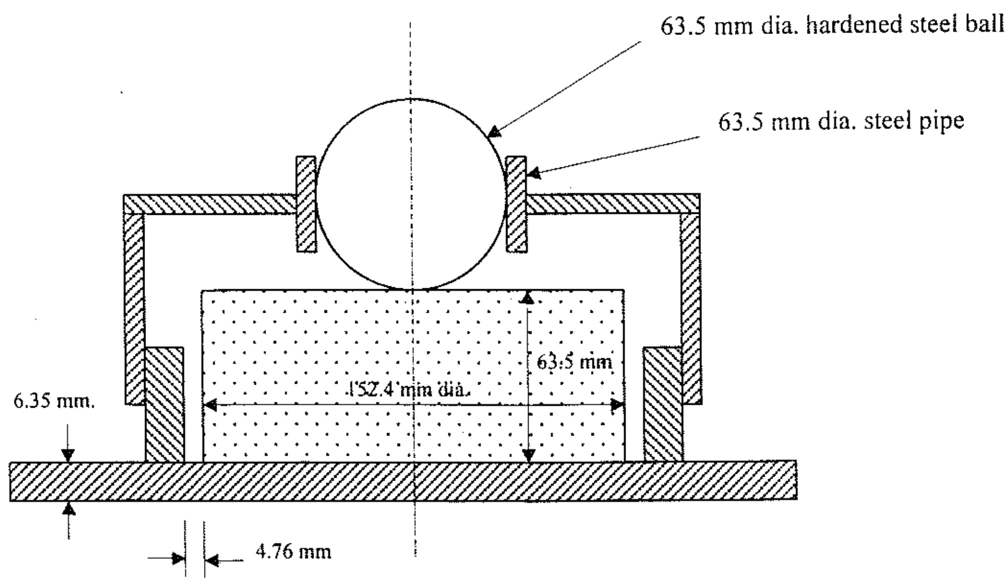

Figure 6. Impact resistance test set-up (3).

RESTRAINED DRYING SHRINKAGE(Provisional test no Specs Assigned)

Restraint of drying shrinkage movements in hardened concrete is a common cause of cracking in concrete systems. In concrete pavements drying shrinkage cracking of concrete can be due to subbase friction or it can occur in a full-depth patch from the restraint provided by dowel bars.

Large concrete surface areas can easily undergo drying shrinkage cracks, so it is hard in the laboratory to reproduce actual restrained drying shrinkage conditions. In the laboratory, ring-type specimens are as shown in Figure 7.

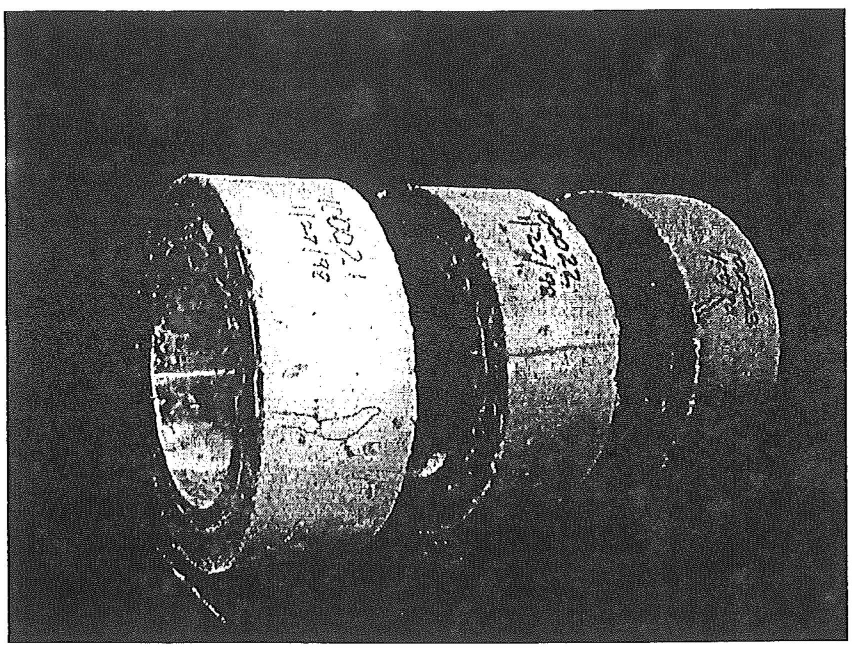

Figure 7. Ring Specimens

The specimens are cast in two layers. After 24 hours the outer mold is removed exposing the concrete surface. The top and bottom faces of the rings are covered with silicone rubber sealer to prevent any moisture movements other than through circumferential area.

The specimens are then exposed to air at approximately 23^{0}C and 40% R.H. The steel ring inside the concrete specimen restrains the concrete specimen causing it to shrink. This restraint will develop internal tangential tensile stresses, which causes concrete to crack. The width and length (area) of these cracks will then represent the damaging effect. The dimensions of the ring tests specimen are shown in Figure 8.

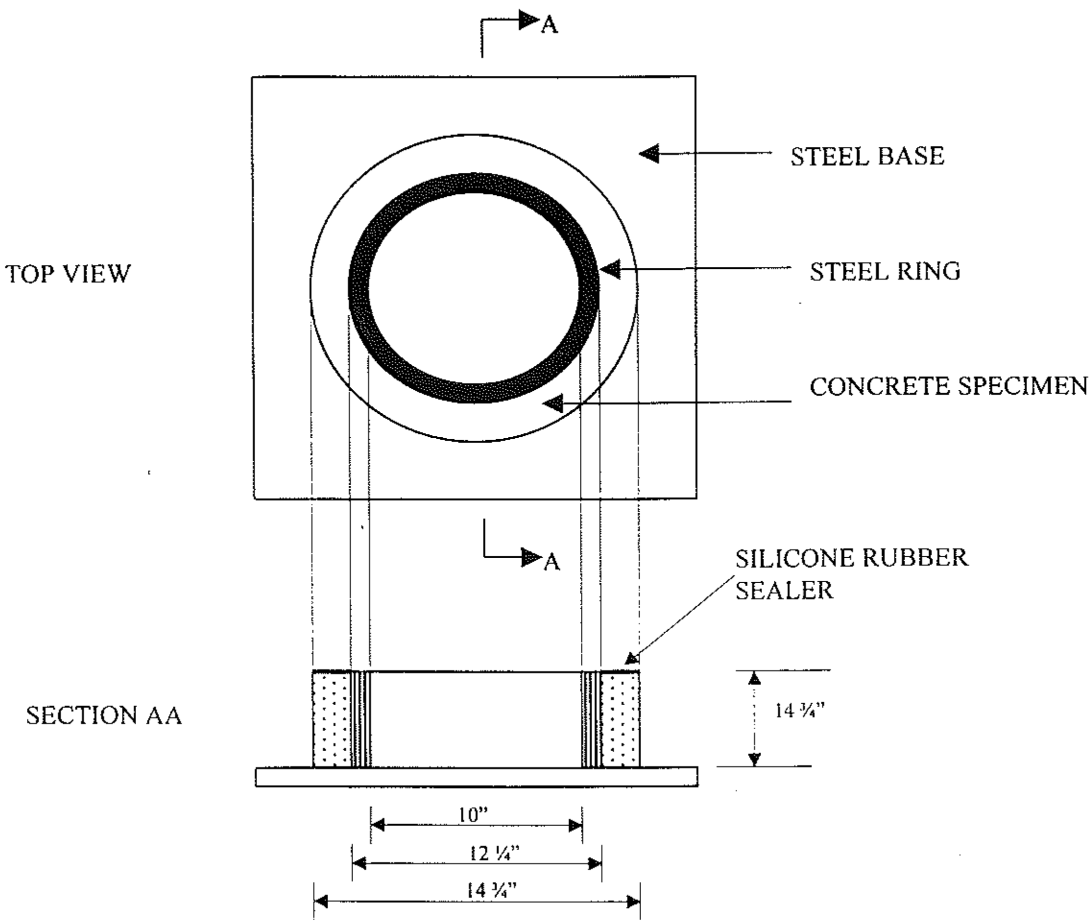

Figure 8. Restrained Drying Shrinkage Test Ring (2).

MECHANICAL PROPERTIES-FIELD SAMPLES

The field molded concrete specimens were cured on site for 24 hours and then returned to an environmental chamber for subsequent curing. The environmental chamber was at 100% relative humidity and 73°F temperature. The specimens were under these controlled conditions until appropriate test ages were reached. In general, all specimens were tested at 3, 7, 14, 28, and 56 days.

Compressive Strength

Table 4 and Figure 9 summarize the compressive strength test data (in accordance with ASTM C39-86 "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens). The target 28-day compressive strength for this mixture design (as supplied by the contractor, Table 2) was 24 MPa.

Table 4. Compressive Strength Results

Time	Sample 1	Sample 2	Sample 3	Average*
0	0	0	0	0
3	22.05	21.56	21.75	21.79
7	26.43	26.08	26.21	26.24
14	28.08	26.83	27.24	27.39
21	28.97	28.47	28.97	28.80
28	30.24	31.38	31.72	31.11
56	32.76	33.10	33.28	33.05

*Measured in Mpa.

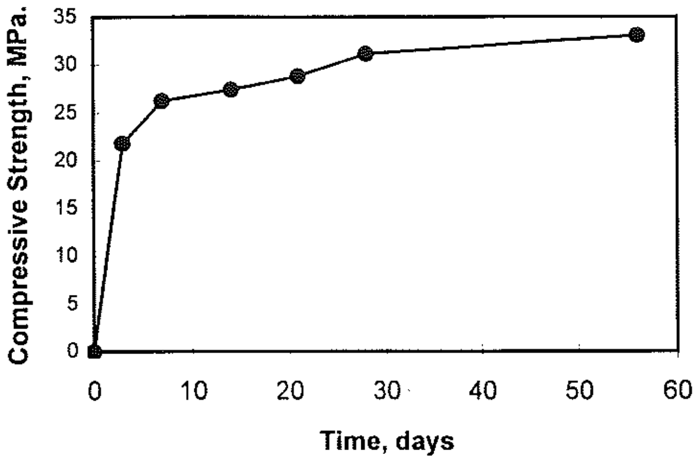

Figure 9. Compressive Strength Gain as a Function of Time.

The laboratory strength of the field-molded cylinders was in excess of 30 MPa. Approximately 85% of the 28-day strength was developed in the first seven days of curing. The contractor specified concrete mixture was replicated in the laboratory and Figure 10 illustrates the compressive strength gain for that mixture.

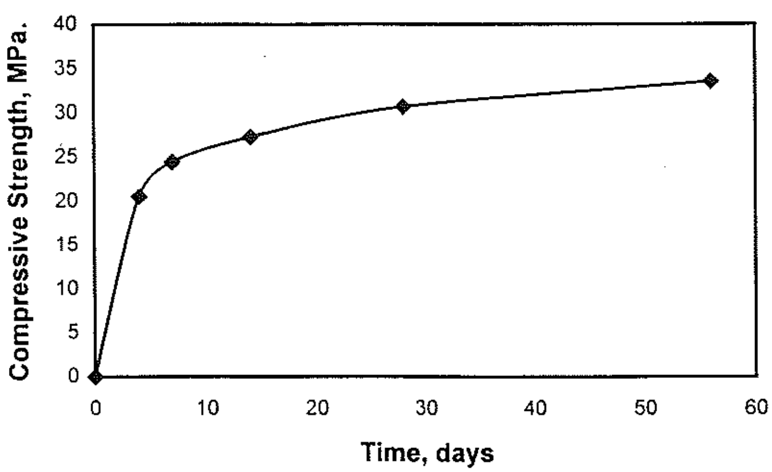

Figure 10. Compressive Strength Gain as a Function of Time for the Laboratory Replicated Mixture.

Flexural Strength

Table 5 and Figures 11 and 12 summarize the flexural strength test data (third point loading). The test was performed in accordance with ASTM C78-84 "Standard Test Method for Flexural Strength of Concrete. A target 28-day flexural strength for this mixture design was not specified. A visual examination of the beams after testing revealed excellent aggregate interlock across the crack faces.

Table 5. Flexural Strength Data, in MPa.

Time, days	Sample 1	Sample2	Sample 3	Average
0	0.00	0.00	0.00	0.00
3	3.12	3.18	3.14	3.15
7	3.63	3.66	3.50	3.60
14	3.70	3.75	3.62	3.69
28	4.10	3.80	4.00	3.97
56	4.20	4.00	4.30	4.17

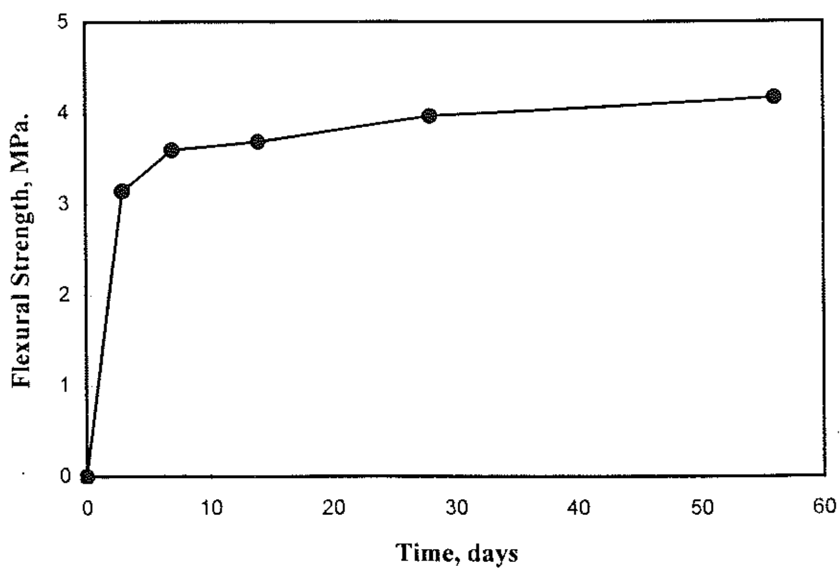

Figure 11. Flexural Strength gain as a Function of Time.

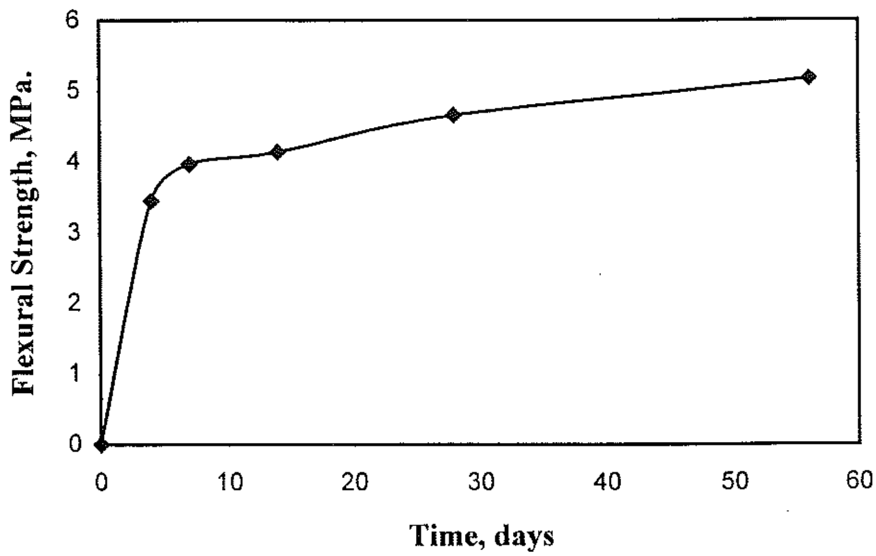

Figure 12. **Flexural Strength Gain as a Function of Time for the Laboratory Replicated Mixture.**

Utilization of a strong and angular large coarse aggregate resulted in a greater percentage of the fracture area, thus creating a ball and socket configuration.

Indirect Tensile Strength (IDT)

The IDT specimens were 150 mm. in diameter and 300 mm. in length and the load was transmitted through 2 bearing strips (made out of wood) 3.2 mm thick, 25 mm wide and marginally longer than the specimen length (in accordance with ASTM C496-90, "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens). The stress distribution schematic is illustrated in Figure 13 and the test results are illustrated in Figure 14. Equation 3 was used for calculating the tensile stress.

$$\sigma_t = \frac{2P}{\pi \bullet L \bullet D} \quad (3)$$

where:

σ_t	=	horizontal tensile stress;
P	=	applied compressive load;
L	=	length of specimen; and
D	=	diameter of specimen.

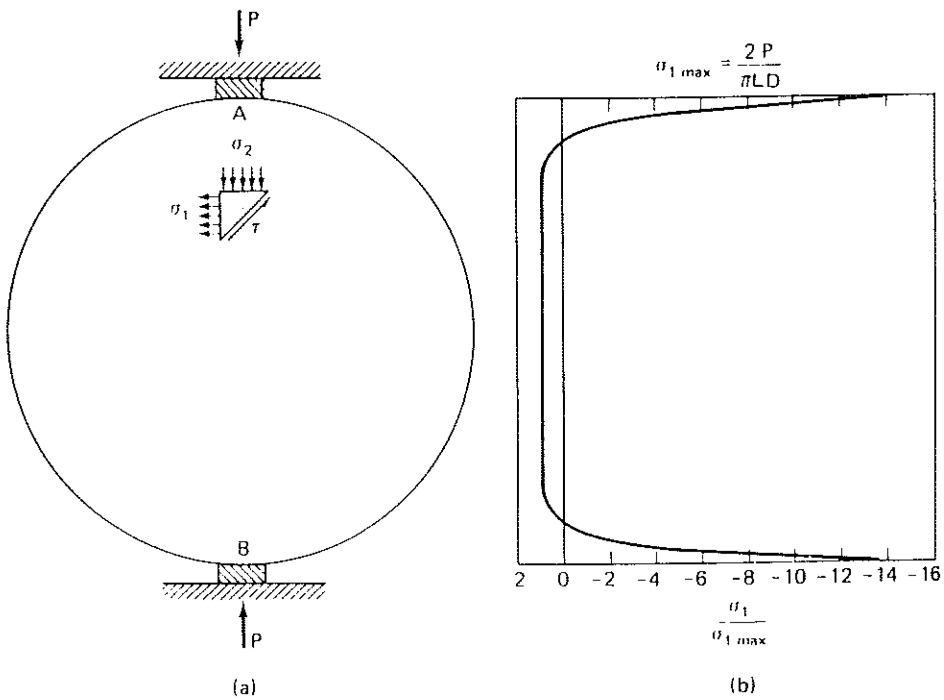

Figure 13. Stress Distribution across Loaded Diameter for a Cylinder Compressed between two flat plates.

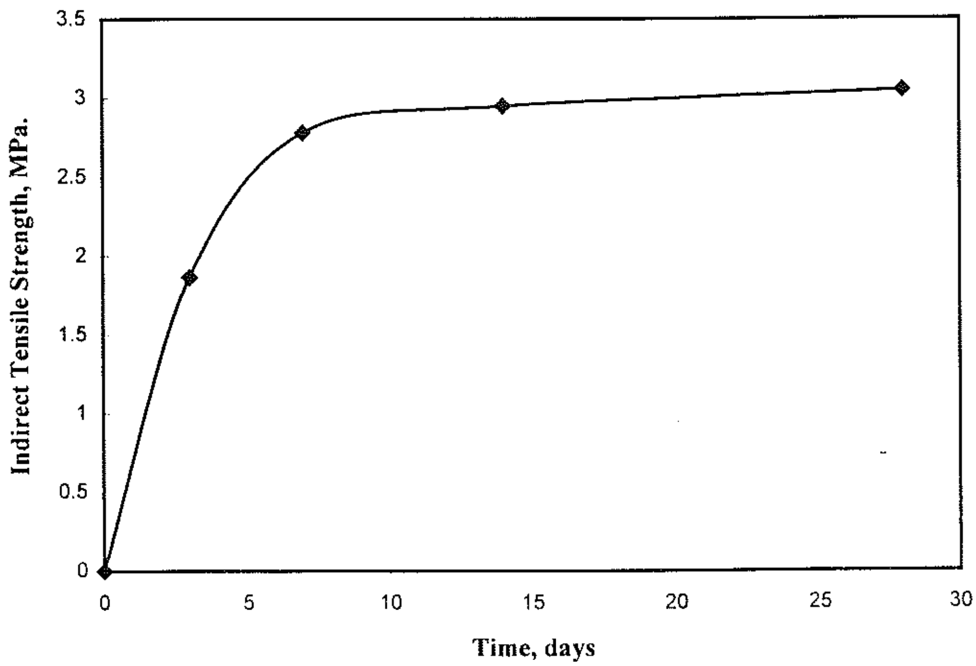

Figure 14. IDT Test Results.

Impact Testing

Plain concrete is brittle in nature, which is one of the biggest drawbacks of using plain concrete in pavements, which are subjected to various types of impact loads. Impact resistance measures the dynamic energy absorption as well as strength of the concrete. The test procedure as adopted by ACI Committee 544.2R-89 was explained in the previous sections of this report. Table 6 and Figure 15 summarize the impact test results of the field-molded samples. The results are expressed in number of blows to first crack.

Table 6. Impact Test Results.

Time, days	Sample 1	Sample 2	Sample 3	Average
0	0	0	0	0
3	7	15	13	12
7	21	28	25	25
14	31	36	41	36
21	29	48	48	42
28	51	44	48	48
56	60	50	49	53

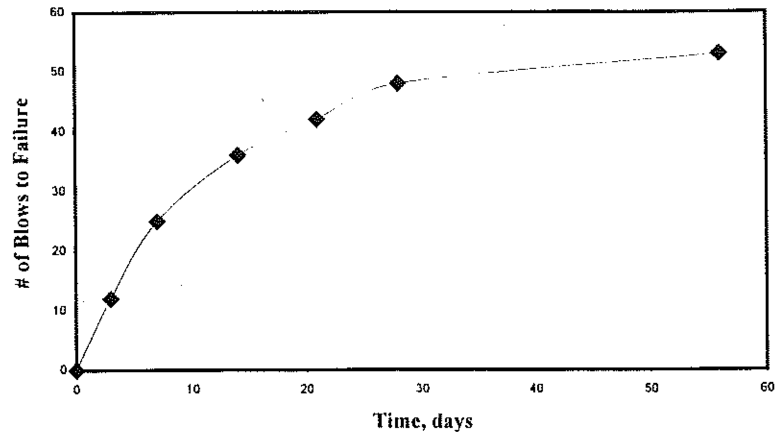

Figure 15. Impact Test Results.

MECHANICAL PROPERTIES OF PCC CORES

The Michigan Department of Transportation (MDOT) extracted 18 cores to determine as-built slab depth. 16 cores were turned over to the research team to conduct mechanical property tests on the as placed concrete. The results of the core tests are presented below in Table 7

Table 7 summarizes the as-built thickness of the PCC slab. Four thickness measurements were made (one on each quadrant) and then averaged. As can be seen from the summary statistics presented in Table 7 the within the core standard deviation and coefficient of variation are very small. The overall average core thickness was 13.11" with a standard deviation of 0.826" and a coefficient of variance of 6.3%.

Table 7. Summary of Core Data Measurements.

Specimen No.	Length (1)	Length (2)	Length (3)	Length (4)	Avg. Length	Std. Dev	COV
4	13.500	13.563	13.750	13.375	13.547	0.156	1.15%
5	12.125	12.250	12.250	12.063	12.172	0.094	0.77%
6	14.250	14.188	14.250	14.250	14.234	0.031	0.22%
7	13.250	13.438	13.250	13.125	13.266	0.129	0.97%
8	14.375	14.313	14.188	14.250	14.281	0.081	0.56%
9	12.375	12.438	12.313	12.500	12.406	0.081	0.65%
10	12.438	12.438	12.375	12.500	12.438	0.051	0.41%
11	13.750	13.438	13.625	13.750	13.641	0.148	1.08%
12	14.438	14.813	14.625	14.625	14.625	0.153	1.05%

13	13.000	13.188	13.000	12.938	13.031	0.108	0.83%
16	12.688	12.875	12.875	12.938	12.844	0.108	0.84%
17	13.438	13.500	13.438	13.438	13.453	0.031	0.23%
18	12.375	12.313	12.438	12.250	12.344	0.081	0.65%
19	12.500	12.500	12.438	12.500	12.484	0.031	0.25%
20	13.250	13.438	13.313	13.250	13.313	0.088	0.66%
21	11.875	11.688	11.750	11.813	11.781	0.081	0.68%
Average	*13.102*	*13.148*	*13.117*	*13.098*	***13.116***		
Standard Deviation	*0.815*	*0.852*	*0.830*	*0.824*	***0.826***		
COV	*6.22%*	*6.48%*	*6.33%*	*6.29%*	***6.30%***		

Note: All measurements are in inches.

Table 8 summarizes the 28-day IDT data for the cores. 5 cores were tested with 2 replicates. The average IDT strength for the 10 samples was 3.04 MPa. as compared to 3.05 Mpa. IDT strength of the field molded samples.

Table 8. IDT Strength Data for PCC Cores.

Specimen #	Sample 1	Sample 2	Average	Std. Dev	COV
9	3.25	3.02	3.14	0.16	5%
10	3.11	3.03	3.07	0.06	2%
11	3.19	3.04	3.12	0.11	3%
12	2.83	2.61	2.72	0.16	6%
13	3.09	3.20	3.15	0.08	2%
Average	3.09	2.98	3.04		
Std. Dev	0.16	0.22	0.18		
COV	5%	7%	6%		

Table 9 summarizes the 28-day compressive strength test results for the PCC cores extracted from the field. The average compressive strength for the 10 samples was 31.23 MPa. as compared to 28.5 MPa. compressive strength of the field molded samples.

Table 9. Compressive Strength Data for PCC Cores.

Specimen #	Sample 1	Sample 2	Average	Std. Dev	COV
4	29.28	27	28.14	1.61	6%
5	32.36	29.22	30.79	2.22	7%
6	26.78	28	27.39	0.86	3%
7	29	26	27.50	2.12	8%
8	29.28	27.9	28.59	0.98	3%
Average	29.34	27.62	28.48		

Std. Dev	1.99	1.20	1.38		
COV	7%	4%	5%		

PROJECT 1: I-75 IN WAYNE COUNTY

The construction boundaries for this project extended from milepost 4.578 to milepost 7.000 in Wayne County, Metro region (IM 82194-36005A). The construction involved the reconstruction of the existing freeway mainline, shoulders, and ramps. The mainline pavement (both in the north and south directions) was reconstructed as a 11" jointed plain portland cement concrete pavement resting on an existing 16" subbase (4" OGDC, 4G with a geotextile separator). The concrete mixture designs for this project is summarized in Table 10.

Table 10. Concrete Mixture Design (as provided by the contractor).

Mixture Ingredients	**I-75 (South)* Mass, kg. (SSD)**	**I-75 (North)* Mass, kg. (SSD)**
Cement	267.40	310.30
Sand, Pit#63-48	880.80	894.50
Fly Ash (Type F)	42.50	-
Coarse Agg 1 (4AA) Pit#75-5	381.70	381.70
Coarse Agg 2 (6AAA) Pit#75-5	579.40	579.40
Water	138.10	138.10
AEA, Axim Catexol AE-260	98.51mL	98.51mL

*All weights are in kg/m^3

The bulk specific gravity of the coarse aggregate (dolomitic limestone) 4AA and 6AAA was 2.656 and 2.637 respectively. The coarse aggregate 4AA and 6AAA were blended in a 40-60 ratio. The specific gravity of the fine aggregate was not reported on the concrete mixture design report. The grain size distribution is illustrated in Figures 16 through 18. The physical properties of the blend are summarized in Table 3.

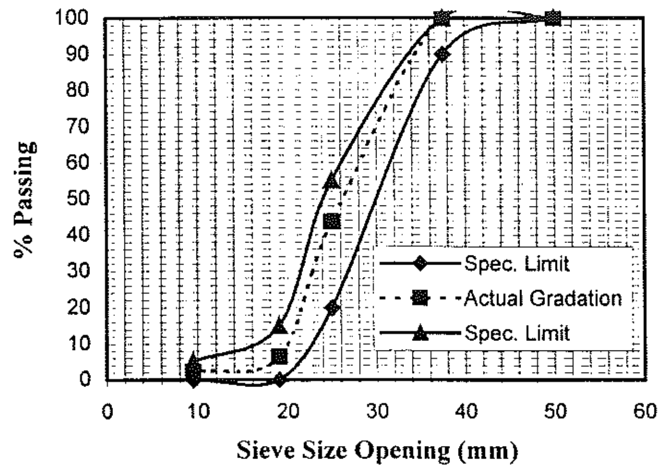

Figure 16. **Particle Size Distribution for 4AA.**

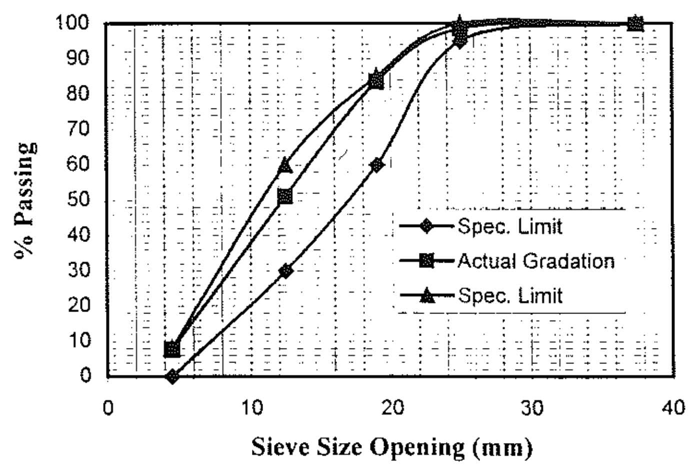

Figure 17. **Particle Size Distribution for 6AAA.**

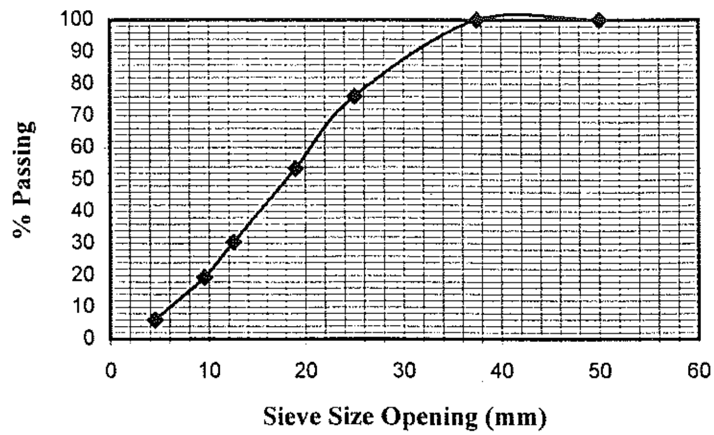

Figure 18. Particle Size Distribution 40%(4AA)-60%(6AAA).

MECHANICAL PROPERTIES-FIELD SAMPLES

The field molded concrete specimens were cured on site for 24 hours and then returned to an environmental chamber for subsequent curing. The environmental chamber was at 100% relative humidity and 73°F temperature. The specimens were under these controlled conditions until appropriate test ages were reached. In general, all specimens were tested at 3, 7, 14, 28, and 56 days.

Compressive Strength

Tables 11a and 11b and Figure 19 summarize the compressive strength test data (in accordance with ASTM C39-86 "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens). The target 28-day compressive strength for this mixture design (as supplied by the contractor) was 24 MPa.

Table 11a. Compressive Strength Results (North Bound), in Mpa.

Time	Sample 1	Sample 2	Sample 3	Average
0	0	0	0	0
1	22.07	23.72	24.00	23.26
4	24.83	25.86	25.34	25.34
8	28.83	30.17	28.34	29.11
14	30.58	30.68	31.00	30.75

28	32.49	34.43	33.00	33.31
56	36.00	37.00	36.40	36.47

Table 11b. Compressive Strength Results (South Bound), in Mpa.

Time	Sample 1	Sample 2	Sample 3	Average
0	0	0	0	0
1	23.41	22.74	24.68	23.61
4	24.78	26.00	23.00	24.59
8	32.70	32.33	30.00	31.68
14	36.41	34.50	33.00	34.64
28	39.71	38.12	36.00	37.94
56	42.00	40.00	41.00	41.00

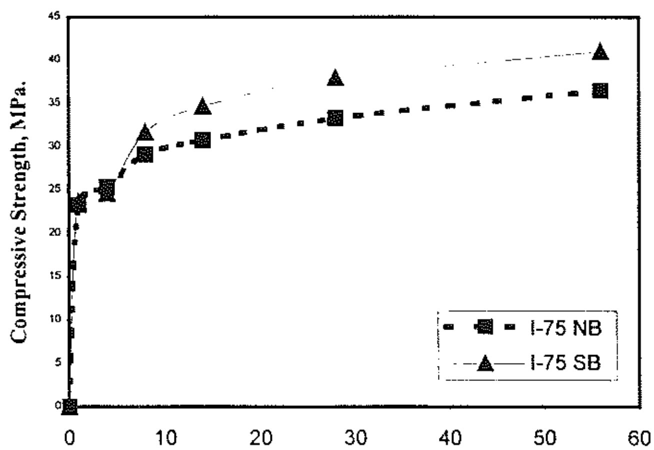

Figure 19. Compressive Strength Gain as a function of Time.

Flexural Strength

Tables 12a and 12b and Figure 20 summarize the flexural strength test data (third point loading). The test was performed in accordance with ASTM C78-84 "Standard Test Method for Flexural Strength of Concrete.. No target 28-day flexural strength for this mixture design was specified. It appears that the fly ash concrete mixture (south bound lanes) develops a higher late age strength when compared to the mixture design used for the north bound lanes.

Table 12a. **Flexural Strength Data (North Bound), in Mpa.**

Time	Sample 1	Sample 2	Average
0	0	0	0
1	2.9	2.6	2.75
4	3.2	3.3	3.25
8	3.7	3.5	3.6
14	4.1	4.4	4.25
28	4.6	5	4.8
56	5.1	5.3	5.2

Table 12b. **Flexural Strength Data (South Bound), in Mpa.**

Time	Sample 1	Sample 2	Average
0	0	0	0
1	2.3	2.6	2.45
4	3	3.3	3.15
8	4.1	3.9	4
14	4.5	4.1	4.3
28	4.8	4.5	4.65
56	5.3	5.1	5.2

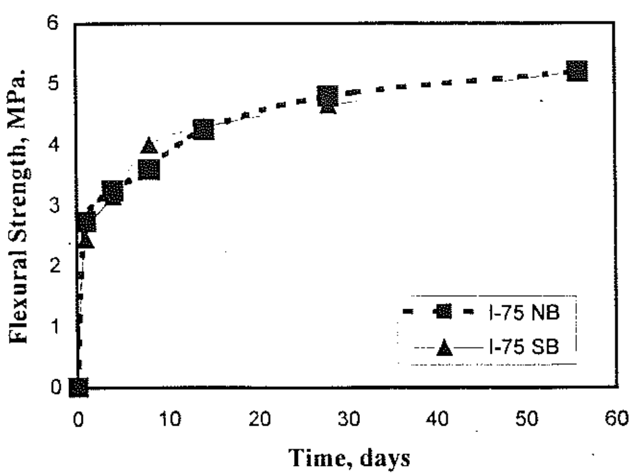

Figure 20. Flexural Strength Data.

Indirect Tensile Strength

The IDT specimens were 150 mm. in diameter and 300 mm. in length and the load was transmitted through 2 bearing strips (made out of wood) 3.2 mm thick, 25 mm wide and marginally longer than the specimen length (in accordance with ASTM C496-90, "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens). The test data is illustrated in Figure 21

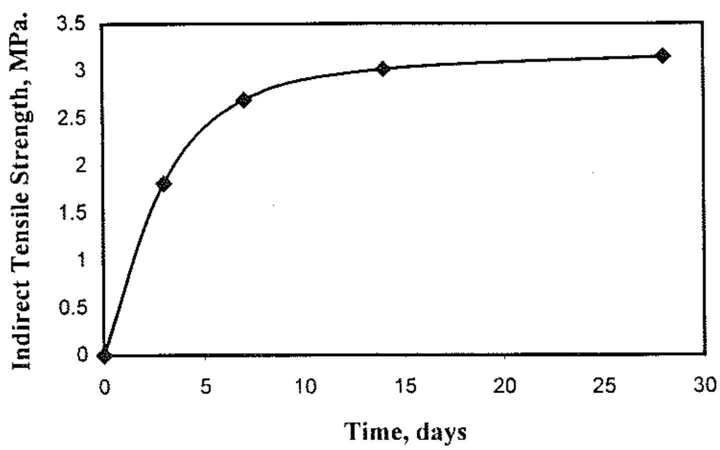

Figure 21. IDT Test Data.

Impact Testing

Plain concrete is brittle in nature, which is one of the biggest drawbacks of using plain concrete in pavements, which are subjected to various types of impact loads. The test procedure as adopted by ACI Committee 544.2R-89 was explained in the previous sections of this report. Tables 13a and 13b and Figure 22 summarize the impact test results of the field-molded samples. The results are expressed in number of blows to first crack.

Table 13a. Impact Test Data (North Bound)

Time, days	Sample 1	Sample 2	Sample 3	Average
8	8	11	7	9
14	23	21	28	24
21	36	41	35	37
28	46	41	39	42

Table 13b. Impact Test Data (South Bound)

Time, days	Sample 1	Sample 2	Sample 3	Average
3	5	13	8	9
8	14	9	11	11
14	23	28	18	23
28	45	18	42	35

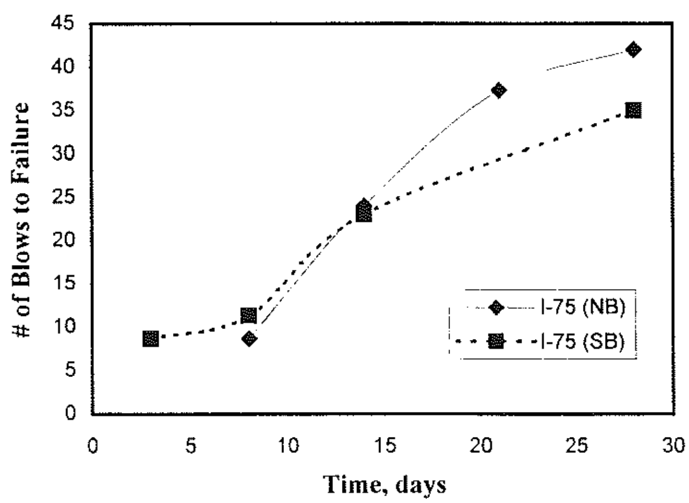

Figure 22. Impact Strength Test Data.

MECHANICAL PROPERTIES OF PCC CORES

The Michigan Department of Transportation (MDOT) extracted 22 cores to determine as-built slab depth. 6 cores were turned over to the research team to conduct mechanical property tests on the as placed concrete. The physical measurements are presented in Table 14 and the mechanical property test results of the core samples are presented in the subsequent sections below. The average core depth was 318 mm with a standard deviation of 9 mm

Table 14. Summary of Field Core Data (as provided by MDOT).

Specimen No.	Depth, mm	Depth, inches
1	325.000	12.795
2	323.000	12.717
3	318.000	12.520
4	320.000	12.598
5	314.000	12.362
6	330.000	12.992
7	310.000	12.205
8	300.000	11.811
9	324.000	12.756
10	330.000	12.992
11	324.000	12.756
12	322.000	12.677
13	320.000	12.598
14	301.000	11.850
15	320.000	12.598
16	310.000	12.205
17	307.000	12.087
18	302.000	11.890
19	316.000	12.441
20	335.000	13.189
21	314.000	12.362
22	315.000	12.402
Average	*318.188*	*12.527*
Standard Deviation	*9.013*	*0.355*
COV	*2.83%*	*2.83%*

Table 15 summarizes the 29-day compressive strength test results for the PCC cores extracted from the field. The average compressive strength for the 9 samples ranges from 34 MPa. to 37 MPa. as compared to 38 MPa. compressive strength of the field molded samples.

Table 15. Compressive Strength Data (29-days) for PCC Cores.

Specimen Number	Sample 1	Sample 2	Sample 3	Average
1	34.63	37.00	35.93	35.85
2	38.23	35.95	37.35	37.18
3	32.54	33.89	36.00	34.14

Table 16 summarizes the 28-day IDT data for the cores. 5 cores were tested with 2 replicates. The average IDT strength for the 9 samples was 3.02 MPa. as compared to 3 Mpa. IDT strength of the field molded samples.

Table 16. IDT Strength Data (29-days) for PCC Cores.

Specimen Number	Sample 1	Sample 2	Sample 3	Average
1	3.40	3.60	2.98	3.33
2	2.67	3.00	2.57	2.75
3	3.30	3.1	2.68	2.99

DURABILITY PROPERTIES

Table 17 summarizes the results from the ring restrained shrinkage test (this is still a provisional test and is not a standard test method adopted by AASHTO or ASTM). The test results are presented in the form of crack area versus the drying period after casting. The results suggest that the standard concrete mixture (MDOT 35-P) is prone to early cracking when compared to the large stone mixture. This is in line with the hypothesis that large stone mixtures require a lesser amount of cement and hence its potential to exhibit shrinkage cracks is greatly reduced. The large stone mixtures exhibited no cracks after 12 days, whereas, the control mixture rings developed cracks after 10 days of placement. After three months, the large stone mixture exhibited a cracked area which was 62% less than that of a standard mixture. Similar trends were observed at the 10 and 30-day intervals. It is worth noting that the variability associated with this test is high, however it provides a basis for comparison on a relative scale rather than an absolute scale.

Table 17. Restrained Shrinkage Data

Mix Type	Sample No.	# of Cracks	First Crack(days)
Standard Mixture	1	4	10
	2	6	11
	3	5	7
Average		5	9.33
Std. Deviation		1	2.08

		Cracked Area (mm^2)		
Mix Type	Sample No.	10 days	30 days	90 days
Standard Mixture	1	0	13.17	19
	2	0	13.35	22
	3	7.23	20.45	33.45
Average		2.41	15.66	24.82
Std. Deviation		4.17	4.15	7.63

Mix Type	Sample No.	# of Cracks	First Crack(days)
Large Stone	1	4	12
	2	2	11
	3	4	13
Average		3.33	12.00
Std. Deviation		1.15	1.00

		Cracked Area (mm^2)		
Mix Type	Sample No.	10 days	30 days	90 days
Large Stone	1	0	8.42	9.9
	2	0	0	9.8
	3	0	7.86	8.2
Average		0.00	5.43	9.30
Std. Deviation		0.00	4.71	0.95

Formation of micro-cracks at a very early age due to plastic shrinkage cracking is one of the main reasons of premature concrete deterioration. Reduction in plastic shrinkage cracking due to the reduced cement content can prove to be very beneficial for the long term performance of concrete pavements. Table 18 summarizes the plastic shrinkage data. It is clear from the cracking ratio that the large stone mixture exhibited only a 29% area cracked when compared to the standard concrete mixture. This is in line with the hypothesis that large stone mixtures require a lesser amount of cement and therefore its potential to exhibit shrinkage cracks is greatly reduced.

Table 18. Plastic Shrinkage Data (110°F,40% R.H.)

Mix Type	Standard Mixture	Large Stone Mixture
Cracking value (mm^2)	3.502	1.208
	3.632	0.989
	3.838	1.124
	3.610	0.978
Mean	3.65	1.07
Std. Dev	0.140	0.111
Crack Ratio	100.000	29.483

Chapter 3

Conclusions and Future Research Needs

The preliminary investigation shows that utilizing the largest possible top size coarse aggregate in a portland cement concrete mixture is an important component for producing a highly durable, and cost efficient concrete pavement. The large coarse aggregate mixtures not only enhance the mechanical properties of the mixture, but also permit the use of a lesser quantity of cementitious material, than currently specified. Based on the field and laboratory data presented in this report, the following conclusions were made:

- The laboratory tested physical properties of the coarse aggregates were similar to those reported by the contractor. The 4AA and 6AAA gradations were in accordance with the MDOT specifications. Based on the laboratory testing for a fineness modulus (fine aggregates) of 2.80 and a maximum coarse aggregate size of 50 mm, the b/b_o was 0.74.
- It is evident that large coarse aggregate concrete mixtures exhibit adequate consistency and mechanical properties and present no major construction problems. No conclusive inferences about the in place durability characteristics can be made at this time due to the short in-service life of the pavements.
- The compressive strength properties of both field molded and laboratory molded specimens were well above the contractor's target 28-day strength.
- There is considerable agreement between the mechanical properties of the extracted cores and the field and laboratory molded specimens.
- A visual examination of the fractured beams after testing revealed excellent aggregate interlock across the crack faces. Utilization of a strong and angular coarse aggregate

resulted in a greater percentage of fracture area, thus creating a ball and socket configuration. This configuration is considered to be a very important mechanism toward maintaining integrity for transferring a wheel load across a pavement crack.

- The relative impact properties of both field samples and laboratory-molded samples appear to be adequate.
- It was clear that the addition of type "F" fly-ash impacted the 28- and 56-day mechanical properties when compared with non-fly-ash mixtures.

Future Research Needs

- It is strongly recommended that an on-going (on a yearly basis) monitoring program be initiated to monitor both the mechanical and durability properties of the concrete mixture.
- For the I-75 Monroe project a comparison can be made between the large coarse aggregate mixtures used in the right truck lane and the traditional full-depth concrete mixtures, since extensive full depth patches have been placed in the middle and median left lanes.
- In order to test the premise that a reduction in cement content and addition of pozzolanic improves the volumetric properties (shrinkage and creep) of the mixture extensive plastic, and drying shrinkage tests need to be performed.
- A Volumetric Surface Texture Analysis needs to be performed on extracted PCC cores from large coarse aggregate mixture pavements to determine the distribution of macro- and micro-texture and whether this can be related to load transfer efficiency across a crack.

Chapter IV

References

1. Goodspeed, C., Vanikar, S., and Cook, R., "High Performance Concrete Defined for Highway Structures: Version 1.0," October 1995.
2. ASTM C.09.03.04 Task Group on Shrinkage Testing Draft 6 June 1992, " Standard Test Method for Evaluating Cracking of Restrained Fiber Reinforced Concrete".
3. ACI 544.2R-89 "Measurement of Properties of Fiber Reinforced Concrete" reported by ACI Committee 544.

Appendix

Plastic Shrinkage Test Procedure

Plastic Shrinkage Cracking of Restrained Fiber-Reinforced Concrete

ANTONIO NANNI, DENNIS A. LUDWIG, AND MICHAEL T. MCGILLIS

It is well established that the low-volume addition of fibers (synthetic or steel) to concrete can significantly reduce cracking due to plastic shrinkage. However, as of today, there is no consensus standard test method available to measure this effect. A test procedure is under scrutiny at ASTM. In this procedure, the surface cracks of a plain concrete panel are compared with those of a fiber-reinforced concrete panel under conditions of severe and controlled moisture loss. The results of an experimental project aiming to determine the validity and repeatability of the test procedure on plastic shrinkage under consideration by ASTM are reported. For this purpose, various fiber types (i.e., synthetics and steel), in different configurations (i.e., monofilament, fibrillated, deformed) and of various lengths, were used with the same concrete matrix. The results show that the proposed ASTM standard has merit. Its major drawback is that specimen performance characterization is based exclusively on crack width.

Volume changes of fresh concrete are due to water absorption and evaporation, sedimentation and segregation, cement hydration, and thermal changes. In addition to the mixture constituents and proportions, volume changes are influenced by the surrounding environment (i.e., temperature, humidity, and wind speed). Plastic shrinkage cracking occurs in the superficial layer of fresh concrete within a few hours after placement. The principal cause of this type of cracking is an excessively rapid evaporation of water from the concrete surface, such that it exceeds the rate at which bleeding water rises to the surface (*1,2*). The formation of plastic shrinkage cracking takes place when internal stress is higher than the tensile strength of concrete. Internal stress is closely related to the capillary pressure of the pore water within the fresh concrete (*1*). Plastic shrinkage cracking occurs most often in slabs and pavement construction exposed to hot and dry weather. Construction operations (screeding and finishing) have a very significant effect on plastic shrinkage cracking (*3*). Cracking can be avoided with the proper concrete mixture design and the proper construction and curing procedure.

In recent years, the use of fibers, particularly of the synthetic type, has become common to minimize plastic and early drying shrinkage cracking in slab-on-grade construction. From here, the need has emerged for a testing procedure that would quantify the beneficial effects of fiber addition and could help in selecting the most appropriate fiber parameters (i.e., fiber type, length, volume percentage) for a specific concrete matrix subjected to specific environmental conditions. Several test methods have been proposed (*4–6*). In 1985, Kraai mentioned in the introduction to his paper that a testing procedure was under consideration by ASTM (*4*). But 7 years later, no standard test has been approved. A task group within ASTM Subcommittee C09.03.04 has arrived at the fifth draft of a proposed method for evaluating plastic shrinkage cracking of restrained fiber-reinforced concrete (FRC). No data have been published in the literature on the performance of this proposed test method other than a summary diagram presented by Berke et al. (*6*). The diagram shows only the average crack area for nine mixtures using the same fiber type at three different lengths and three different volumes. The salient feature of the test procedure being considered is in the specimen configuration (see Figure 1). In this case, the restraining effect of a perimeter wire mesh as proposed by Kraai (*4*) is substituted with three stress risers. Shrinkage cracking is expected to initiate at the central riser, where the specimen thickness is reduced from 100 mm (4 in.) to 38 mm (1.5 in.). The experimental results obtained by Berke et al. (*6*) with this specimen are very similar to those obtained by Kraai and presented by Vondran and Webster (*7*). Significant shrinkage cracking reduction was obtained when using fibers in the concrete matrix.

The objective of this research project was to evaluate independently the validity and reliability of the proposed ASTM procedure by obtaining several experimental results on plain concrete and FRC mixtures.

TEST PROGRAM

Materials

The concrete matrix used for the entire project had the following proportions: portland cement Type I, 335 kg/m^3; water, 208 kg/m^3; coarse aggregate (20 mm maximum size), 1037 kg/m^3; fine aggregate, 814 kg/m^3. This mixture had a very high cement content and a very high water-cement ratio, and it did not contain any chemical or mineral admixtures. The slump was 180 mm (7 in.), the unit weight was 2286 kg/m^3 (3,853 lb/yd^3), and the air content was 0.9 percent. The 28-day compressive strength was 24.7 MPa (3,590 psi) with a standard deviation of 2.5 MPa (361 psi) for 15 specimens.

The fibers used in the testing program are described in Table 1; their sources are not identified. Two types of steel fibers were used from two different manufacturers. Three mixtures (B, C, and D) contained steel fibers (straight and deformed) at the low dosage of 0.62 percent by weight (corresponding to 25 lb/yd^3). The remaining nine FRC mixtures were made with synthetic fibers of two base materials and different configurations (monofilament and fibrillated). The

Department of Architectural Engineering, Pennsylvania State University, 104 Engineering Unit A, University Park, Pa. 16802.

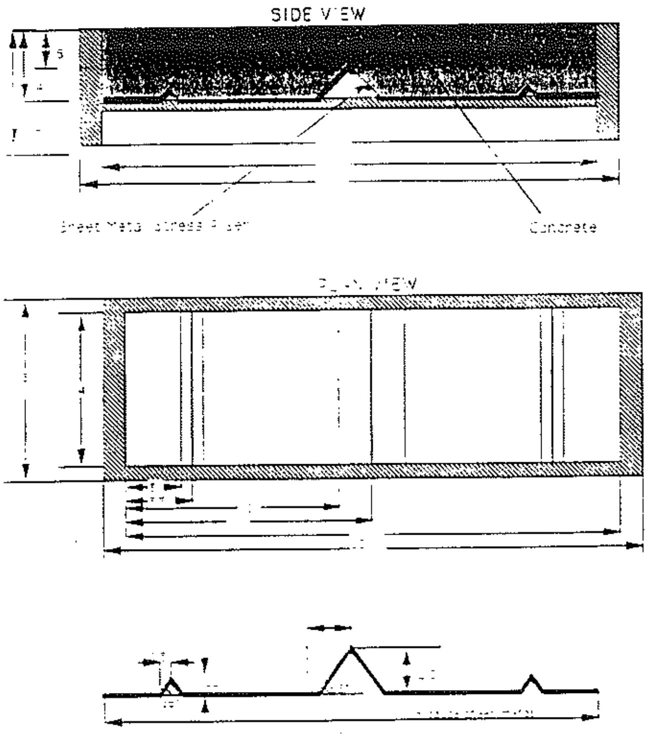

FIGURE 1 Specimen geometry and configuration (ASTM proposed standard) (not to scale).

TABLE 1 Specimen Key

MIXTURE	FIBER-MANUFACTURER	VOLUME (%)	CHARACTERISTICS
A	Plain Matrix	--	--
B	Steel-I	0.10	Straight
C	Steel-II	0.10	Deformed
D	Steel-I	0.10	Deformed
E	Synthetic-I	0.05	Monofilament
F	Synthetic-I	0.05	Monofilament
G	Synthetic-I	0.05	Monofilament
H	Synthetic-II	0.10	Fibrillated
I	Synthetic-II	0.10	Fibrillated
J	Synthetic-II	0.10	Fibrillated
K	Synthetic-II	0.10	Fibrillated
L	Synthetic-II	0.10	Fibrillated
M	Synthetic-II	0.07	Monofil

first type was used in different lengths and at the same dosage in three mixtures (E, F, and G). The second type was used in six mixtures (H, I, J, K, L, and M) with different lengths and at different dosages.

Fabrication and Testing Procedures

Each testing day 0.16 m^3 (5.5 ft^3) of concrete was batched. First, gravel and sand stored in sealed containers were placed into the mixer. While the aggregates were being weighed, the specimen molds were lightly oiled for bond breaking and preventing water absorption along the wooden sides of the molds. At the same time, predetermined amounts of fibers were prepared. After the aggregates were placed in the mixer, the mixer was turned on and half of the mixing water was added. Following a 3-min premixing, portland cement and, afterward, the rest of the mixing water were added. The concrete was then left to mix for another 5 min. A known amount of concrete was then discharged from the mixer. Part of this amount was used for the plain matrix specimen, compression cylinders, and fresh concrete tests (slump and air content). At this point, the selected fiber type was added to the concrete remaining in the mixer and mixed for an additional 3 min. This amount of FRC was sufficient for two identical specimens. After discharging the first FRC batch, the mixer was reloaded with an equal amount of plain matrix and the second fiber type was added for two more FRC specimens.

Five molds were filled with concrete and vibrated on a vibratory table for 12 sec and then screeded, floated, and weighed. The specimens were moved to an environmentally controlled room to be placed into individual air ducts. The room was equipped with a thermostat and a dehumidifier. At the start of each day, the room temperature was kept at approximately 38°C (100°F) and the relative humidity between 25 and 30 percent. Water pans were filled and weighed and then placed in each duct on a weighing scale positioned beside the specimen (so that the evaporation from the specimen would not interfere with the evaporation from the pan, and vice versa). The fans were turned on, pushing air across the specimens. Initial readings were taken (air and concrete temperature, humidity, and wind speed). Subsequent readings were taken every 30 min for 3 hr (for a total of seven readings). At the completion of the 3 hr period, the fans were turned off, a final water pan weight reading was taken, and the specimens were removed from the ducts. The specimens were then weighed and the length and width of cracks were measured. The width of each crack was measured with a crack scale at approximately every inch along its length. The crack length was determined by placing a string along the crack and then measuring the length of the string. The average width and the length were multiplied to compute the area of one crack. This procedure was repeated for each crack, and the total crack area was calculated by summing up individual values. After all measurements were taken, the specimens were disposed of.

With respect to the proposed ASTM test procedure, some comments that could lead to future improvements are offered:

- The procedure for filling the mold should be clearly spelled out. To the operator, it is natural to place scoops of concrete to the left and right of the central stress riser and then distribute the material over the entire mold. If this is done, the number of fibers crossing the riser may not be representative of the nominal fiber volume in the mixture.
- The water pan should be placed beside the specimen rather than behind it. The water pan placed on the scale in the wind stream can wobble and spill easily. If the pan is filled according to the proposed standard, the water will blow out of the pan.
- The specimens are large and unwieldy even for two people.

DISCUSSION OF RESULTS

Environmental Conditions

The proposed test procedure does not specify the environmental conditions in terms of temperature and relative humidity, but it specifies a minimum air flow velocity of 4.5 m/sec (10 mph). Any combination of these three parameters is suitable, provided that the evaporation rate in the water pan is at least 980 g/m²/hr (0.2 lb/ft²/hr). To satisfy this requirement, it was attempted to maintain environmental conditions inside the duct as close as possible to 35°C (95°F), 40 percent relative humidity, and a wind speed of 5.2 m/sec (11.5 mph). The crack area of plain matrix specimens can be plotted as a function of the average temperature, relative humidity, and wind speed recorded during the 3-hr test. In this case, it appears that matrix cracking is insensitive to environmental conditions within the ranges experienced during this project. Even the combination of the three independent variables in one single parameter (directly proportional to temperature and wind speed and inversely proportional to relative humidity) has no effect on cracking area. It is therefore concluded that, as long as the evaporation rate in the water pan remains close to the prescribed value of 98 g/m²/hr, no significant effect on cracking is expected due to slight changes in environmental conditions.

The first shrinkage crack in all specimens was visible at the fifth (120 min) or sixth (150 min) interval reading. No significant difference in cracking time between plain matrix and FRC was observed. The average concrete temperature at the beginning of the test was 21.9°C (71.4°F) with a standard deviation of 1.1°C (1.9°F). After 3 hr, at the end of the test the average concrete temperature had climbed to 30.8°C (87.4°F) with a standard deviation of 2.9°C (5.2°F). The average room temperature and relative humidity over a 3-hr period were 36.8°C (98.2°F) and 38 percent.

Evaporation from Specimens

The addition of fibers to concrete has been reported to decrease the amount of water bleeding (7). The weight loss of all specimens was measured at the end of the 3-hr test. The crack area of all specimens can be plotted as a function of the weight loss (i.e., evaporated water). In this case, the trend of the data points would indicate that the higher the moisture loss, the higher the crack area. In addition, data points relative to the unreinforced matrix tend to cluster at the upper-right side of the diagram corresponding to higher values of evaporation and cracking. It can be concluded that, in general, water evaporation in FRC is less (and with less cracking) than for the respective plain matrix.

Plain Concrete and FRC Cracking

All specimens (plain matrix and FRC) cracked during the performance of the test. Given the specimen geometry and configuration, once a crack started over the central stress riser, it usually extended over the full width of the specimen and, obviously, could grow no further. The parameter that characterizes the performance of different samples becomes, therefore, the width of the crack. This is demonstrated by the two diagrams in Figure 2. In this diagram, the crack area is plotted as a function of the crack width for all specimens. The crack width given here represents the average value of all measurements for each sample. The first observation is that average crack width varied widely between 0.1 and 1 mm (one order of magnitude between maximum and minimum values), whereas the total length of cracks was about 380 mm (15 in.) (the width of the specimen was 356 mm, or 14 in.). The second observation is that the data points relative to the matrix concentrate at the higher values of the abscissa (wider cracks). It is concluded that for the specimen size and configuration of the proposed test method, the paramount characterization parameter is crack width.

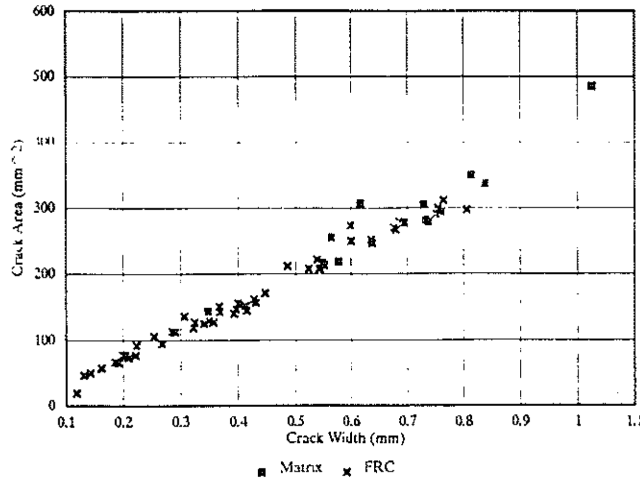

FIGURE 2 Crack area versus crack width (all specimens).

Evaluation of Proposed ASTM Standard

The average crack area of each FRC mixture (four samples), expressed as a percentage of the companion plain matrix specimen, is shown in Figure 3. In this diagram, the sample standard deviation is also plotted above and below the average value. From this figure, it is observed that with the exception of FRC Mixture F, the shrinkage cracking area of any FRC

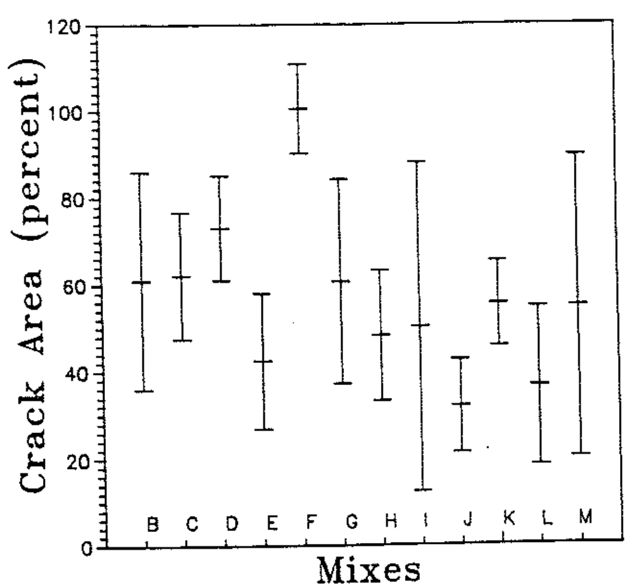

FIGURE 3 Average crack area (and standard deviation) for each FRC mixture relative to respective plain matrix.

is lower than that of the companion matrix. The variability is rather high but may be attributed to the nature of the parameter under study (shrinkage cracking) rather than the procedure itself.

Looking at the results from a practical end, assume that this project was intended to identify the most suitable fiber types to reinforce a given matrix. Would the proposed test method have helped? The answer is probably yes. The engineer could set an acceptability threshold (say, 50 percent shrinkage crack area reduction), consider all fiber types with better performance, and use this information with other parameters (e.g., cost, workability, etc.) to select the most desirable product. What is missing is the verification of the test results in terms of field performance. For this, only time and more work can provide the answer.

CONCLUSIONS

The objective of this work was to generate experimental data with a proposed ASTM standard test method meant to evaluate the ability of fibers to control plastic shrinkage cracking. The authors have concluded that the proposed method has merit and is not irremediably flawed. The major concern is in the fact that, because of the specimen configuration, crack width becomes the primary parameter to characterize the shrinkage cracking potential of different specimens. Rather than continuing an endless discussion, it is probably in the best interest of the public and the fiber industry to adopt a standard test method for the evaluation of plastic shrinkage cracking. The method can then be reevaluated after a fixed period.

ACKNOWLEDGMENTS

Financial contributions and materials were provided by steel and synthetic fiber manufacturers.

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Publication of this paper sponsored by Committee on Mechanical Properties of Concrete.