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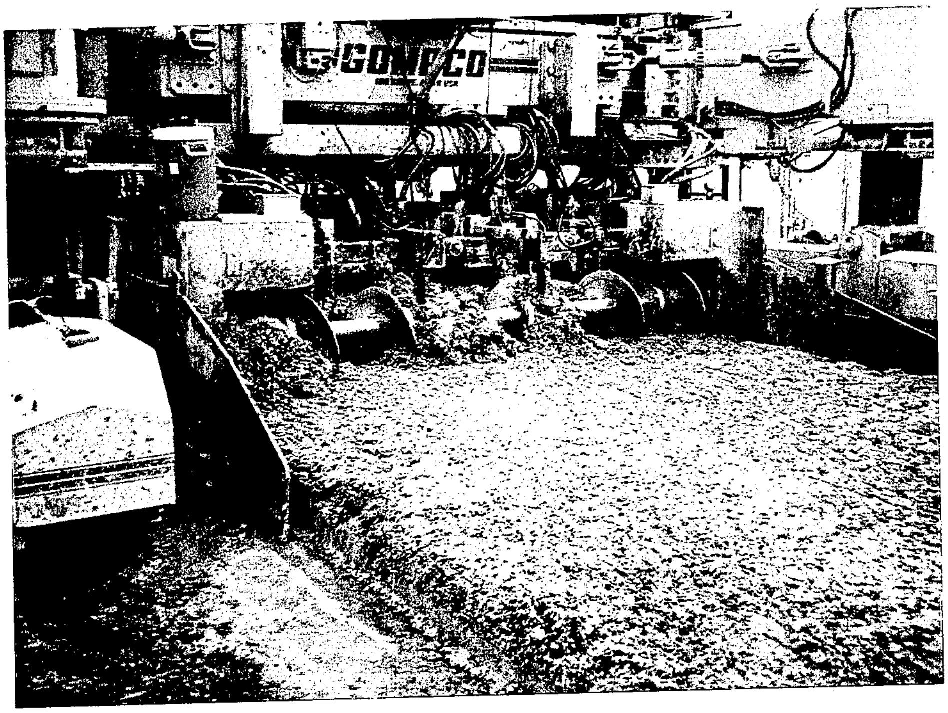

EVALUATION OF NON-METALLIC FIBER REINFORCED CONCRETE IN NEW FULL DEPTH PCC PAVEMENTS

Study SD96-15
Final Report

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16. Abstract

This final report presents the construction and performance evaluation of a new full depth pavement, constructed with a new type non-metallic fiber reinforced concrete (NMFRC).

The mixture proportions used, the quality control tests conducted for the evaluation of the fresh and hardened concrete properties, the procedure used for mixing, transporting, placing, consolidating, finishing, tining and curing of the concrete are described. Periodic inspection of the full depth pavement was done and this report includes the results of these inspections.

The feasibility of using this NMFRC in the construction of highway structures has been discussed. The new NMFRC with enhanced fatigue, impact resistance, modulus of rupture, ductility and toughness properties is suitable for the construction of full depth pavements. However, a life-cycle cost analysis shows that NMFRC is not a favorable choice, because of it's high initial cost.

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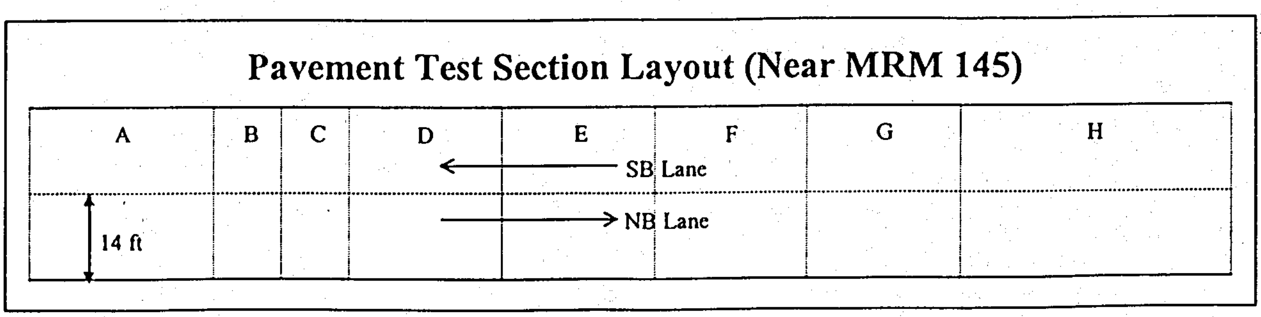
EXECUTIVE SUMMARY

The performance of a relatively small NMFRC full depth test section, constructed as a part of SDDOT's research project SD 94-04 *Evaluation of Non-Metallic Fiber Reinforced Concrete in PCC Pavements and Structures*, performed favorably. Before NMFRC's use in full depth pavement is accepted, the following problems must be addressed: 1. The constructability and economic impacts of using these fibers must be determined in order to support its continued use, 2. Design criteria must be established to determine pavement thickness, joint spacing, etc., 3. The effectiveness of load transfer across joints, should be studied further, 4. The formation and spacing of the random cracks must be determined, and the behavior of jointed and unjointed slabs must be understood. Therefore there was a need for this research in order to find answers to the above stated problems.

The research objectives were

1. To recommend NMFRC full-depth pavement designs that will enhance PCC performance.
2. To evaluate constructability and performance of NMFRC full depth pavement.
3. To evaluate the economic impacts of using NMFRC full depth pavement.

The project involved the construction of full depth NMFRC and control test sections on US 83 northeast of Pierre, South Dakota between mileage reference markers (MRM) 144 and 145. Two lanes, each 14 ft. wide of the following test sections as shown in the figure below were constructed:



A: Doweled Plain Jointed Concrete Pavement (PJCP) control section 305m (1000ft) long, 200mm (8 inch) thick, with 6.1m (20 ft.) joint spacing.

B: Undoweled NMFRC test section 76m (250 ft) long, 165 mm (6.5 inch) thick, with 7.6m (25 ft.) joint spacing.

C: Undoweled NMFRC test section 75m (245 ft) long, 165 mm (6.5 inch) thick, with 10.7m (35 ft.) joint spacing.

D: Doweled NMFRC test section 152m (500 ft) long, 203 mm (8 inch) thick, with 7.6m (25 ft.) joint spacing.

E: Doweled NMFRC test section149 m (490 ft) long, 203 mm (8 inch) thick, with 10.7m (35 ft.) joint spacing.

F: Undoweled NMFRC test section 152m (500 ft) long, 203 mm (8 inch) thick, with 7.6m (25 ft.) joint spacing.

G: Undoweled NMFRC test section 149m (490 ft) long, 165 mm (6.5 inch) thick, with 10.7m (35 ft.) joint spacing.

H: NMFRC test section 390m (1290 ft) long, 203 mm (8 inch) thick, with no joints.

The research activities involved were to review and summarize literature relevant to FRC in full depth pavement applications, develop NMFRC mix proportions, conduct quality control testing, recommend construction methods, and monitor and evaluate the test sections. The research activities also involved periodic condition surveys to evaluate the performance of the constructed pavements. It was also proposed to compare the performance of this new NMFRC, with the plain concrete pavement.

The test program on fresh concrete included slump, concrete temperature, fiber content, air content, vebe time and unit weight. The hardened concrete properties included: compressive strength, static modulus, modulus of rupture, load-deflection curves, first crack toughness strength and post crack behavior, ASTM toughness indices, Japanese toughness index, equivalent flexural strength, fatigue strength, and impact strength. The mixture proportions used, the procedure used for mixing, transporting, placing, consolidating, finishing, and curing during the construction of highway pavement are described.

The Polyolefin fibers incorporated in the concrete at a rate of 14.8 kg/m^3 (25 lb./cu.yd.) performed well in the mixing operation without causing any balling or segregation. However in the beginning, there were some bundles that did not open causing the fibers not to disperse. The problem was corrected by prewetting the fibers and slightly increasing the mixing time. The fresh concrete properties tested during construction were found satisfactory. The mean 28-day compressive strengths of concrete placed on August 15, 1996 were 31.23 Mpa (4530 psi) and 30.44 Mpa (4415 psi) slightly higher than the specified compressive strength of 27.6 Mpa (4000 psi). The 28-day compressive strengths of concrete placed on August 26, 1996 were 28.10 Mpa (4075 psi) and 34.85 Mpa (5055 psi). However the addition of polyolefin fibers at 14.8 kg/m^3 (25 lbs./cu.yd.) enhanced the structural properties of concrete. There was a considerable increase in toughness, impact, fatigue, endurance limit, and post crack load carrying capacity. The most important contribution due to the addition of fibers to concrete is the change in the mode of failure from a dangerous brittle failure to a more desirable ductile failure when subjected to compression, flexure, impact and fatigue loads. The toughness indices showed an increase in elasto-plastic behavior of the concrete in comparison to plain concrete.

The feasibility of using NMFRC in the construction of full depth pavement has been established. The same construction techniques and construction equipment without any permanent modification could be used in construction of full depth pavements using NMFRC. However, the plant was slightly modifidied by mounting a large diameter plastic pipe into opening between store and weigh bins, so that the fibers could be added to the batching process. Also the number of people at the plant had to be increased to add the fibers.

Periodic inspection of the newly constructed pavement was made. P.K. nails were placed across the joints by the D.O.T. immediately after paving. The distances between the P.K. nails were recorded after placing and during the nine periodic inspections. As expected, random cracks occurred in the unjointed test section. These cracks formed at approximately 26 m (85-ft) intervals and they were almost straight transverse cracks, continuous in both north and south bound lanes. They appeared to be similar to the regular sawed joints. No random cracks were found in any of the other NMFRC or control sections. There were no joint spalling, raveling, cracking, and pop-outs in both control and NMFRC pavements.

There was no consistent pattern shown in the P.K. nail measurements and hence no positive conclusions could be made based on these measurements. The inspections had shown that there was no difference in the behavior of the joints and joint cracks for the thicker (203 mm or 8 in.) pavement and thinner (165mm or 6.5 in.) pavements.

It is possible to have longer joint spacings in NMFRC pavements. In the short duration of 3 years, the inspections had shown that there was no distress such as excessive cracking, spalling and fatigue cracking at the joints in pavement segments with 7.6m (25-ft) joint spacings and 10.6m (35-ft) joint spacings.

The post construction performance of the control and NMFRC pavement section was satisfactory. Once the cracks formed in the unjointed NMFRC pavement section, the polyolefin fibers helped to contain the crack propagation and to resist the widening of cracks.

The Falling Weight Deflectometer Test and tests for International Roughness Index (IRI) and South Dakota Index (SDI) were conducted by SDDOT engineers and the results were supplied. The Falling Weight Deflectometer Test has shown that the load transfer was less in all the NMFRC sections compared to the control section. In general the load transfer was less in sections with longer joint spacings, less thickness, and undoweled. However the test had also shown that the elastic modulus value for NMFRC was 15 percent less than that of the control concrete with the same strength. This result is contradictory to all the known facts and therefore the results are not totally acceptable.

No difference in the riding quality could be established between PCC and NMFRC pavements with different joint spacings, different thicknesses, and doweled and undoweled sections. The measured International Roughness Index (IRI) and South Dakota Index (SDI) for both control and NMFRC pavements were satisfactory for the new pavement criteria. There was no significant difference in these indices for the control section and various sections of the NMFRC pavement.

The life cycle cost analysis has shown that NMFRC with its high initial cost is not a favorable material for the construction of full depth pavements. However when the cost of fiber becomes less expensive, then this may be a viable material for the construction of full-depth pavement.

Based on the results and observations in this project, the following recommendations are made:

1. When initial cost is not a deciding factor, and when longer joint spacings, thinner sections and more efficient performance are the requirements, then NMFRC full depth pavements could be used in special cases.
2. The equations and empirical constants used in the evaluation of load transfer in the Falling Weight Deflectometer Test should be analyzed and modified for use with NMFRC pavement so that they reflect correctly the known facts about the concrete modulus values.
3. A more accurate method could be used for measuring crack-widths and joint openings, and their changes with seasons and with time.
4. Since no other design procedure is available currently, it is recommended that the NMFRC pavement thickness can be designed by modifying the PCA pavement thickness design method using appropriate NMFRC test data.
5. It is recommended that the following control tests be conducted for NMFRC fresh concrete: slump, unit weight, air content, and fiber content.
6. It is recommended, that the concrete temperature, the ambient temperature, humidity, and the wind velocity be recorded during the placing of the concrete.
7. It is recommended that field samples be collected and cured using ASTM standard procedures and that the following hardened concrete performance tests be conducted for NMFRC at 28–days: compressive strength, elastic modulus, flexural strength (modulus of rupture), fatigue strength, and toughness values(ASTM and Japanese standards).
8. It is recommended that the same construction procedures for mixing, transporting, placing, consolidating, finishing, tining, and curing used for full depth paving with plain concrete be used for NMFRC. NOTE: Some additional mixing time may be required for NMFRC which should be determined by field trials.
9. Based on the observed short term performance of the joints in the NMFRC, it is recommended that longer joint spacings could be used for NMFRC pavement. For thicker full depth NMFRC pavements (203mm (8.0in)) a 15.25m(50 ft) joint spacing is suggested, and for thinner full depth NMFRC pavements (165mm (6.5in)) a 10.7m (35ft) joint spacing is recommended. This recommendation is based on the

assumption that the FWD data did not truly reflect the load transfer in NMFRC pavements.

10. It is recommended that SDDOT's standard saw depths used for transverse and longitudinal joints be used for NMFRC full depth paving.
11. Based on the FWD results, it is recommended that control joints for NMFRC full depth pavement use dowel baskets as a mehtod to transfer load across joints.

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GLOSSARY

The following is a glossary of terms for fiber reinforced concrete (FRC) used in this report.

0.1 General Terms

Aspect Ratio - The ratio of length to diameter of the fiber. Diameter may be equivalent diameter.

Balling - When fibers entangle into large clumps or balls in a concrete mixture.

Collated - Fiber bundled together either by cross-linking or by chemical or mechanical means.

Equivalent Diameter - Diameter of a circle with an area equal to the cross-sectional area of the fiber.

Fiber content - The weight of fibers in a unit volume of concrete.

Fibrillated - A fiber with branching fibrils.

First Crack - The point on the flexural load-deflection or tensile load-extension curve at which the form of the curve first becomes nonlinear.

Hairline Crack – Cracks with widths less than 0.1 mm (0.0039 inches) are termed as hairline cracks.

First Crack Deflection - The deflection value on the load deflection curve at the first crack.

First Crack Strength - The stress obtained when the load corresponding to first crack is inserted in the formula for modulus of rupture given in ASTM Test Method C 78.

First Crack Toughness - The energy equivalent to the area of the load deflection curve up to the first crack.

Flexural Toughness - The area under the flexural load-deflection curve obtained from a static test of a specimen up to a specified deflection. It is an indication of the energy absorption capability of a material.

Toughness Indices - The numbers obtained by dividing the area under the load-deflection curve up to a specified deflection by the area under the load-deflection curve up to "First Crack" as given in ASTM C 1018.

Toughness Index, I_5 - The number obtained by dividing the area up to 3.0 times the first crack deflection by the area up to the first crack of the load deflection curve, as given in ASTM C 1018.

Toughness Index, I_{10} - The number obtained by dividing the area up to 5.5 times the first crack deflection by the area up to the first crack of the load deflection curve, as given in ASTM C 1018

Toughness Index, I_{20} - The number obtained by dividing the area up to 10.5 times the first crack deflection by the area up to the first crack of the load deflection curve, as given in ASTM C 1018

Residual Strength Factor $R_{5,10}$ - The number obtained by calculating the value of $20(I_{10}-I_5)$, as given in ASTM C 1018.

Residual Strength Factor $R_{10,20}$ - The number obtained by calculating the value of $10(I_{20}-I_{10})$, as given in ASTM C 1018.

Flexural Toughness Factor (JCI) - The energy required to deflect the fiber reinforced concrete beam to a mid point deflection of 1/150 of its span.

Equivalent Flexural Strength (JCI) - It is defined by

$$F_c = T_b \times s/\delta_{tb} \times b \times d^2$$

where

F_c = equivalent flexural strength, psi

T_b = flexural toughness, inch-lb

s = span, inches

δ_{tb} = deflection of 1/150 of the span, inches

b = breadth at the failed cross-section, inches

d = depth at the failed cross-section, inches

Impact Strength - The total energy required to break a standard test specimen of a specified size under specified impact conditions, as given by ACI Committee 544.

Monofilament - Single filament fiber.

Static Modulus - The value of Young's modulus of elasticity obtained from measuring stress-strain relationships derived from other than dynamic loading.

High Performance Concrete - In this report, High Performance Concrete is defined as a concrete with highly enhanced (or improved) desirable properties for the specific purpose and function for which it is used. It need not necessarily be high-strength concrete. High performance concrete may have one or more of the following properties enhanced: ductility, fatigue strength, durability, impact resistance, toughness, impermeability and wear resistance.

Whitetopping - Whitetopping is concrete placed over asphalt where the concrete thickness is 101 (4 inch) or more mm thick.

Ultra-Thin Whitetopping - Ultra-Thin Whitetopping is concrete placed over asphalt where the concrete is less than 101 mm (4 inch) thick.

0.2 Acronyms Used

ACI - American Concrete Institute

CFP - Collated Fibrillated Polypropylene

FRC - Fiber Reinforced Concrete

LS - Low Slump

NMFRC - Non-Metallic Fiber Reinforced Concrete. This acronym refers only to Polyolefin Fiber Reinforced Concrete. These fibers were manufactured and purchased from 3M for the purpose of this study.

NMFRS - Non-Metallic Fiber Reinforced Shotcrete

PFRC - Polypropylene Fiber Reinforced Concrete

PCC - Portland Cement Concrete

PJCP – Plain Jointed Concrete Pavement

SFRC - Steel Fiber Reinforced Concrete.

SNFRC - Synthetic Fiber Reinforced Concrete

SIFCON - Slurry Infiltrated Fiber Concrete

SIMCON - Slurry Infiltrated Mat Concrete

0.3 ASTM Specifications

A 820 - Specification for Steel Fibers for Fiber Reinforced Concrete

C 31 - Practices for Making and Curing Concrete Test Specimens in the Field

C 39 - Test Method for Compressive Strength of Cylindrical Concrete Specimens

C 78 - Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-point Loading)

C 94 - Specification for Ready-Mixed Concrete

C138 - Test for Unit Weight, Yield and Air Content (gravimetric) of concrete

C 143 - Test Method for Slump of Portland Cement Concrete

C 172 - Method of Sampling Freshly Mixed Concrete

C 173 - Test Method of Air Content of Freshly Mixed Concrete by the Volumetric Method

C 231 - Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method

C 469 - Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression

C 995 - Test Method for Time of Flow of Fiber Reinforced Concrete Through Inverted Slump cone

C1018 - Test Method for Flexural Toughness and First Crack Strength of Fiber - Reinforced Concrete (Using beam with Third-point Loading)

C 1116 - Specification for Fiber Reinforced Concrete and Shotcrete

0.4 International Standards

A - American Concrete Institute Committee 544 Fiber Reinforced Concrete

ACI 544.2R.89 Flexural Fatigue Endurance

Impact Resistance

Toughness

B - British Standards Institute

BS1881: Part 2, Methods of Testing Concrete-Vebe Test

C - Japanese Society of Civil Engineers

JSCE Standard III-1, Specification of Steel Fibers for Concrete, Concrete Library, No. 50, March 1983.

- JSCE-SF4 Standard for Flexural Strength and Flexural Toughness, "Method of Tests for Steel Fiber Reinforced Concrete," *Concrete Library of JSCE,* No. 3, June 1984, Japan Concrete Institute (JCI), pp. 58-66.

- "Standard Test Method for Flexural Strength and Flexural Toughness of Fiber Reinforced Concrete, (Standard SF4),"*JCI Standards for Test Methods of Fiber Reinforced Concrete,* Japan Concrete Institute, 1983, pp. 45-51.

Problem Description

Due to a decaying infrastructure and tightening budget constraints, transportation engineers are being challenged to replace existing PCC pavements economically with an increase in performance. In an attempt to economically increase the performance of South Dakota's highway network, the Department has pursued the use of a new type of fiber reinforced concrete in its PCC pavements and structures.

Technological advances in fiber reinforced concrete (FRC) offer possible solutions to these problems [1 to 4]. Some advantages of FRC appear to be: pavements may be constructed with thinner cross-sections and have performance characteristics and constructability comparable to the thicker non-fiber reinforced concrete; FRC may reduce spalling even in concrete with quartzite aggregate which has a higher thermal coefficient than other aggregate types; joint spacing may be increased due to FRC's enhanced properties, therefore reducing maintenance; 3M's polyolefin fibers have advantages over steel fibers in that they are chemically resistant and have a lower corrosive potential [5 to 24].

SDDOT's research project, SD94-04 *Evaluation of Non-Metallic Fiber Reinforced Concrete in PCC Pavements and Structures,* constructed 4 different test sections using 3M's Polyolefin Fiber System. The test sections were: 1) full depth pavement, 2) bridge deck overlay, 3) Jersey Barrier, and 4) whitetopping. Also the other SDDOT project, *Demonstration of Polyolefin Fiber Reinforced Concrete in a Bridge Deck Replacement, 1995-96,* used 3M's Polyolefin fibers. The test section included, the replacement of the deck slab and both barriers for a bridge across Interstate 90 at Exit 10. Minimal or no additional effort was needed during the construction of these test sections due to the addition of the fibers. The preliminary field inspections show that the non-metallic fiber reinforced concrete (NMFRC) is performing well in each application [5,24]. The improved properties make polyolefin fiber reinforced concrete an attractive material for concrete pavements. Before NMFRC's use in full depth pavements could be accepted, the following items needed to be addressed: the constructability and economic impacts of using these fibers needed to be determined in order to support its continued use; design criteria needed to be established to determine pavement thickness, joint spacing, etc.; the effectiveness of load transfer across joints and random cracks needed to be determined; and the behavior of jointed and unjointed slabs needed to be addressed.

There was an urgent need for the proposed research in order to find answers for the above stated problems. Due to the favorable performance of the relatively small NMFRC test sections, constructed as part of SD94-04, construction of larger full depth pavement test sections in SD96-15 which exhibit full-scale behavior using a fiber addition rate of 15 kg/m^3 (25 lbs/yd^3) answered many of the questions.

Research Objectives

1. To recommend NMFRC full-depth pavement designs that will enhance PCC performance.
2. To evaluate constructability and performance of NMFRC full depth pavement.
3. To evaluate the economic impacts of using NMFRC full depth pavement.

Research Task 1:Meet with Technical Panel to discuss the research topic and work plan.

The Principal Investigator (P.I.) and the Research Associate (R.A.) met with the Technical Panel on June 6, 1996 and discussed and reviewed the project, the procedures, methods and proposed tasks. Valuable suggestions and comments were made by the technical panel. The details about the PK nail installation and methods of measuring the strains were discussed during this meeting. Details about the maturity testing were discussed during the meeting. The tasks to be carried out by the P. I. were discussed.

Research Task 2: Review and summarize literature relevant to FRC in full depth pavement applications

Introduction

The constructed facilities of the world have been deteriorating due to the effect of the natural environment, excessive use beyond the original design, aging of the materials and general obsolescence. FRC composites are almost ideal materials for repair, rehabilitation, retrofit and renovation of the world's deteriorating infrastructure.

Concrete fiber composites technology has grown over the last three decades into a mature industry. Since the pioneering research on steel fiber reinforced concrete conducted in the United States in the 1960's, there has been substantial research and development activities throughout the world [1 to 4].

Most of the early applications of FRC consisted of using relatively high fiber loadings of straight steel fiber, of relatively small diameter and high aspect ratios. These large quantities were needed to obtain higher flexural strengths. This caused serious problems namely "balling" of fibers during the mixing operation and "pull-out" of the straight fiber during loading due to lack of adequate anchorage. Balling of fibers in the mixer prevents uniform distribution and also causes problems when concrete is placed. Because of the pull-out of the fibers, the SFRC did not achieve the required ductility and post-crack load carrying capacity. The addition of straight fibers had to be done by special vibrating sieves or manual sprinkling. The introduction of a new type of fiber (with hooked ends and bundled together with a water soluble glue) had eliminated these problems [2]. Collation of the fibers had eliminated balling and the fibers could be added to the mixer along with the aggregates all at one time eliminating the need for special devices and additional labor for addition of fibers. The improved anchorage (hooked ends) has made it possible for considerably smaller quantities (40 percent less) of fibers to produce the desired properties [2]. The later development of other deformed fibers (corrugated, crimped, paddled, etc.) had similar results of eliminating balling and they had improved anchorage [3].

New Development In Steel Fibers

Using Melt Overflow as a production process in the late 1980's, Ribbon Technology Corporation has recently introduced two new types of steel fibers for concrete. It is claimed [2] that because of mass production these fibers are more cost effective than fibers made from drawn wire and other wrought metal processes. Corrosion resistant 16 mm (5/8 in.) fine fibers are used for crack controlling reinforcement for concrete. So far steel had not been used as such fine fiber. It is claimed [2] that these fibers will perform as well as or better than the polypropylene fiber with the cost equivalent or slightly less than the polypropylene. The high modulus of elasticity of the steel fiber has the advantage over the polypropylene fiber in having less deformations.

The second application of these fibers is in shotcreting and particularly as a stainless steel alloy for shotcreting of thin architectural panels. Thus these fibers could replace the unstable glass fibers currently used for these applications. Higher modulus and unlimited life of the reinforcement are the added advantages compared to glass fibers. These fibers could also be used as supplemental reinforcement of normal steel fiber reinforced concrete and increase the first crack strength significantly.

Another significant development is the production of steel fiber mats in continuous lengths and widths up to 1.22m (4 ft) wide and 50 mm (2 in.) thick. The mats are made of long steel fiber spun directly from the molten metal and air laid on a conveyor system to form an entangled mat of uniform density, thickness and width. These mats are then coiled into a large roll with a carrier material interlaid between the mat layers. These non-woven mats utilize fibers with aspect ratios exceeding 500. Typical aspect ratios currently used range from 40 up to 100 and special handling procedures may be required as the aspect ratio approaches 100. Since the mat is already in the preformed shape, handling problems are minimized and balling does not become a factor.

This steel fiber mat which is a new concept in concrete reinforcement will be better suited for thin shell or thin layer concrete reinforcement such as thin overlays for bridge decks and industrial floors and thin faced tilt wall panels.

New Development In Synthetic Fibers

3M Company, St. Paul, Minnesota, USA, has developed synthetic (polyolefin) fibers with low aspect ratios similar to steel fibers for use in concrete. These fibers would be available in various lengths and diameters and are added to improve the structural properties of concrete like steel fibers. These fibers could be mixed with concrete in large quantities, as much as 20 percent by volume without causing any balling, segregation or increase in air entrainment in concrete. The amount of fibers that could be added depends on the length and diameter of the fibers. However, the performance of these fibers in fresh and hardened concrete depends on the aspect ratio of the fibers. Therefore, it is possible to produce high volume fiber reinforced concrete using the regular concrete mixture proportions including coarse aggregates whereas high volume fiber concrete using steel fibers are produced using cement slurry instead of regular concrete. There are a number of advantages with polyolefin fibers such as no corrosion potential, chemical inertness, and no hazardous or nuisance conditions when fibers become loose or protrude from the concrete surface. Unlike steel fibers, these fibers are nonmagnetic and non-corrosive [3,4].

Fresh concrete properties:

The significant problem with the fiber concrete mixes is that of ensuring adequate workability (flowability and compactability) that will facilitate the concrete to be placed, compacted and finished with ease and also ensuring a uniform fiber distribution. The balling of fibers, segregation of mix, and excessive bleeding during placing and compaction should also be avoided. For a given mix proportion, the degree of compaction seriously affects the strength and other properties. Therefore, the knowledge of fresh concrete properties is essential for proper design of the mix. Adequate information is available about the performance characteristics of the fiber reinforced concrete with and without the use of superplasticizers.

Properties of Hardened Concrete :

The significant influence of incorporation of steel or polyolefin fibers is to delay and control the tensile cracking of the composite material. Thus an inherently unstable tensile crack propagation in concrete is transformed to a slow controlled crack growth. The fibers provide a ductile member in a brittle matrix and the resultant composite has ductile properties which are significantly different from plain concrete.

All modes of failure are affected by fibers. The strengthening mechanism of fibers involves transfer of stress from matrix to the fiber by interfacial shear or by interlock between the fiber and matrix if the fiber surface is deformed. The fiber and matrix share the tensile force until the matrix cracks and then the total force is transferred to the fibers. This change in the mechanism of failure causes significant improvement in the following properties: ductility, toughness, impact resistance, tensile and flexural strengths, fatigue life, abrasion resistance, shrinkage, durability, and cavitation resistance.

Polyolefin Fiber Reinforced Concrete (NMFRC)

An extensive laboratory investigation was conducted at the South Dakota School of Mines and Technology to evaluate the newly developed 3M polyolefin fiber reinforced concrete [6 to 9]. The physical and elastic properties were determined for concrete and mortars reinforced with polyolefin fibers of various diameters ranging from 0.15 to 3.35 mm (0.006 to 0.132 inch), different lengths and shapes. This research had shown that better performance of the fresh and hardened concretes could be achieved by using these fibers in place of steel fibers for certain types of structures such as pavements, thin bridge-deck overlays, full depth bridge decks, and overlays over asphalt pavements (white topping). The overall performance characteristics such as flexural strength, crack growth restraint, toughness (calculated according to both ASTM and Japanese standards), post-crack load carrying ability, the energy absorption capacity to failure, and impact strength were better. Laboratory tests had shown that adding 8.9 Kg/m^3 (15 lbs/cu.yd.) of 0.38 mm (0.015 inch) polyolefin fibers in concrete had given a comparable performance to that of 39 Kg/m^3 (66 lbs/cu. yd.) of best quality steel fibers available in the market. Polyolefin fibers out perform steel fibers at comparable quantities on an equal weight basis, and therefore, are more effective than steel fibers. These fibers have been incorporated in concrete with conventional equipment and procedures.

The proprietary delivery system developed by the 3M Company provides uniform distribution of even higher dosages of fibers into concrete composites without the loss of workability in the fresh concrete. The current synthetic fiber (nylon and polypropylene fibers) loading is typically 0.1 to 0.3 % by volume and steel fiber loading is typically up to 1.0% by volume. The limiting factor in achieving higher loading was the challenge to mechanically incorporate fibers uniformly in concrete. A uniform dispersion of fibers is necessary to maintain rheological properties in the concrete which is necessary for the physical placement of the concrete. Polyolefin fibers 0.63 mm (0.025 inch) diameter and 50 mm (2 inch) length could be added to concrete 1.0 % to 4.0 % by volume of the concrete using the new delivery system. There was no balling or any other difficulty in mixing, placing and consolidating the concrete.[6 to 9]

Several things have been constructed using NMFRC such as a process tower slab, driveways, sidewalk and curb, and overlays over asphalt roads. All placements had been accomplished with conventional equipment and procedures.

Applications of NMFRC (Polyolefin Fibers)

1. Full depth fiber (both steel and polyolefin) reinforced concrete pavement, on Sheridan Lake Road in Rapid City, SD. (SD94-04)
2. A total replacement of the Bridge-deck slab on the bridge at Spearfish, SD (bridge over I-90, exit 10). (SD 95-22)

Construction of Sheridan Lake Road Pavement

As a part of the research program with the SDDOT, the NMFRC pavement at Sheridan Lake Road was constructed next to the existing SFRC pavement, which was constructed in 1992. The NMFRC pavement had a total length of 32.08 m (105 ft.) which included one section of 22.88 m (75 ft.) and two transition sections of 4.6 m (15 ft.) each. The width of the pavement was 14.6 m (48 ft.) The thickness of the section was 140 mm (5-1/2 inches). It was also proposed to keep the proportions of coarse and fine aggregate equal for the NMFRC mixture, resulting in more mortar and paste content of concrete, highly essential for the proper mixing and distribution of the polyolefin fibers. A Steel fiber reinforced concrete (SFRC) pavement was also constructed at the intersection of the Coral Drive and Sheridan Lake Road. Dramix Steel fibers (60 mm long and 0.8 mm dia.) were used.

Laboratory trial mixes were made to determine if the same mixture proportions could be used for both SFRC and NMFRC. Based on the results it was decided to use the same basic mixture proportions for both SFRC and NMFRC and observe the performance of the concrete in regard to workability, fiber mixing efficiency, placement, consolidation and finishing. This experiment showed that with a minor change in fine to coarse aggregate ratio in FRC compared to plain concrete, the fibers mixed well and uniformly distributed throughout the mix without balling or segregation. The uniform mixing was also possible in trucks with a regular load of 6.1 to 6.9 m^3 (8 to 9 cubic yards) of concrete in trucks with 8.4 m^3 (11 cu. yd.) capacity.

Sheridan Lake Road Steel fiber Reinforced Concrete Pavement (SFRC):

Dramix steel fibers (ZC 60/.80) at 39 kg/m^3 (66 lbs/cu.yd.) were added to the concrete at the plant. A concrete paving machine (GOMACO 5800) was used. The vibrators had a capacity of 12,000 revolutions/minute. In the job, the vibrators were used at 8000 to 9000 revolutions/minute. It was specified that the slump of the concrete when it was discharged from the truck should be 25 to 50 mm (1 to 2 inches). However the concrete delivered had slightly higher slump 64 to 89 mm (2-1/2 to 3-1/2 inches). Therefore there was some delay in floating and finishing the pavement surface. There was no difficulty for placing, consolidating and finishing operations by the paver. The fibers were not sticking out and the 'roller-bug' was not used in the finishing operation. The surface was floated similar to the plain concrete. As the paver moved, the concrete surface was dragged with a wet burlap attached to the paver. There were two men finishing, one with a rectangular float and another with a flat float. After the floating, a carpet drag (Astro-turf) was used in the case of plain concrete and a plastic broom was used for fiber concrete for the final finish. In the case of SFRC, a carpet drag (Astro-turf) would have pulled the fibers out of the concrete; hence it was not used. The finish appeared to be satisfactory [5].

Sheridan Lake Road Polyolefin Fiber Reinforced Concrete Pavement (NMFRC):

The fibers used were 50 mm (2 inch long) and 0.63 mm (0.025 inch) diameter, purchased from the 3M Company. The fibers were added at 14.8 kg/m^3 (25 lbs/cu.yd.). The fibers mixed well without balling, segregation or lumping. The fibers were also uniformly distributed. In this project, it was proposed to study the efficiency of the mixing and uniform distribution when the fibers were added in a larger volume of concrete, in the truck. Therefore the fibers were added in the first truck with 3.8 m^3 (5 cu.yd.) of concrete. The second truck contained 5 m^3 (6.5 cu.yd.) of concrete. Subsequent trucks contained 6.1 m^3 (8 cu.yd.) of concrete and the last truck contained 7.3 m^3 (9.5 cu.yd.) of concrete. In all these trucks, the fibers were added at the plant and at the same dosage rate of 14.8 kg/m^3 (25 lbs/cu.yd). The fibers mixed very well and distributed uniformly without any balling, segregation or lumping together. The efficiency of the mixing was maintained even in the truck with 7.3 m^3 (9.5 cu.yd.) of concrete (The capacity of the truck was 11 cu. yd.). It was observed that the fibers were mixed well in the concrete. Any amount of additional vibration or mixing did not induce any balling or segregation or lumping of the fibers. The plain concrete slump delivered at the site was specified as 37 to 50 mm (1-1/2 to 2 inches). At this slump level, the consolidation and finishing of the concrete by the paver was highly satisfactory. For the NMFRC pavement concrete, a lower water/cement ratio (0.41 water to cementitious materials ratio) was used in order to maintain the same strength for both plain and fiber concrete.

The slump of the NMFRC delivered, by the first truck, at the site was 6 mm (1/4 inch). It was decided to retemper the concrete in the field to increase the slump to about 19 to 25 mm (3/4 to 1 inch). The retempering was done with 5 gallons of water in 6.1 m^3 (8 cu.yd.) of concrete. After retempering, the concrete discharged as easily as the plain concrete. The paver operation, and the floating and finishing operation went smoothly, similar to that of the plain concrete. The paver operator and the concrete finishers were fully satisfied with the consistency and finishability of the concrete.

The same mixture proportions were used in the south bound lanes for the second phase. These mixture proportions had given a low workability [about 6 mm (1/4 inch) slump] after addition of fibers, and the concrete had to be retempered in the field with 19 L (5 gallons) of water for 6.1 m^3 (8 cubic yards) of concrete to achieve a slump of 25 mm (1 inch) or more. This procedure worked well and there were no difficulties in discharging, placing and finishing with a paver.

However, it was decided to try out a new procedure to avoid retempering in the field. In this technique, it was decided to add one gallon of superplasticizer for each truck in the plant to get the required workability 25 to 50 mm (1 to 2 inches slump) when the concrete was discharged in the field. This technique was successfully applied for the rest of the concreting.

After comparing both procedures, the authors' preference would be to follow the procedure in which superplasticizer was not used and the workability of the concrete in the field was controlled by retempering with an appropriate amount of water. Care should be taken to insure that the slump after retempering should always be less than the original slump obtained when it was initially mixed.

The joints were saw cut 10 hours after the concrete was placed for plain concrete and SFRC pavements. In the case of NMFRC pavement, the joints were cut, about 4 hours after the placement was finished. The saw cuts were made with the Soff-Cut equipment.

The pavements were inspected after one, two and three days for plastic shrinkage cracking. The inspection revealed that there were no plastic shrinkage cracks in all three pavement sections. These field inspections were made every week for the first three months, to observe the surface conditions for cracks, fiber protrusions, pop-outs, and any other pavement distress. Observations were made on all three sections of the pavements.

The North bound lanes were opened to traffic one week after placement. They were immediately subjected to heavy traffic loads, because all traffic including the concrete ready mix trucks with 10 cubic yards of concrete passed over them during the placement of the two west side lanes.

Some steel fibers were exposed on the surface. However, they were laying flat. Some fibers were shiny at places where they were subjected to wheel traffic. Some polyolefin fibers were also seen at the surface. They were well bonded to the concrete and could not be pulled out. These fibers were not glaringly visible.

Three cracks were observed, one in the plain concrete pavement, one in the drain inlet at the SFRC pavement, and one in the NMFRC section of the pavement. The crack in the plain concrete pavement was 3.22 m (10 ft. 7 inches) long and 3.1 to 1.6 mm (0.124 to 0.0625 inches) wide. It seemed to be a shrinkage crack. The other two cracks in the SFRC and NMFRC had occurred due to the construction of the drainage inlet at these locations. The crack in NMFRC section was a diagonal crack 2.82 m (9 ft. 3 inches) long and 1.5 mm (0.06 inches) wide. It seemed to have occurred due to a settlement or disturbance in the sub-grade during the construction of the drainage inlet. In the SFRC the crack occurred near the storm water drainage. Later inspections showed that the observed cracks remained stable and did not change in size.

Construction of Spearfish Bridge Deck and Barrier:

The project involved a complete deck replacement of a 102 m by 12 m (330' by 40') 30 Degree skew left hand forward, 4 span continuous steel girder structure carrying US 85 over Interstate 90 near Spearfish, South Dakota [24]. The deck thickness was 210 mm (8.25 in.). The mix design was done for a polyolefin fiber addition of 15 kg/m^3 (25 lb./cu.yd) and to satisfy or exceed the SD DOT specifications which require a bridge deck concrete to have a 28 day compressive strength of 4500 psi, an air content of 5.5 to 7.5%, and a slump of 1 to 3.5 inches. There was no difficulty encountered during the mixing, transporting, placing, consolidation and tining of the bridge deck and barrier with this mix. The required or specified workability and finishability was achieved using NMFRC. Ready mixed NMFRC was delivered to the site using mixing trucks. It was then pumped up to the Bridge deck and placed using a bridge deck paver.

The same construction techniques and the same type of construction equipment without any permanent modification were used in the construction of the Jersey barriers using NMFRC. The Jersey barriers had an average area of cross section of 2.26 sq. ft.

Research Task 3:

Propose the testing program, including lab and field tests and field evaluations that will be performed. In addition to the same lab and field tests that were performed in SD94-04, testing and evaluation should identify the behavior of the pavement at joints and uncontrolled cracks. This may include measuring crack widths, falling weight deflectometer (FWD) data, curling measurements, and visual surveys.

The following testing program was proposed.

Quality Control Tests

Tests for Fresh Concrete

The fresh concrete was tested for slump (ASTM C 143), air content (ASTM C 231), fresh concrete unit weight (ASTM C 138) and concrete temperature. The concrete from the unit weight container was washed and the fibers were separated and weighed to determine the actual fiber content in a cubic yard of concrete. The ambient temperature, humidity and wind velocity were also recorded.

Tests for Hardened Concrete

Compressive Strength & Static Modulus

Cylinders were tested for compressive strength at ages 7 and 28 days according to ASTM C 39. Prior to the compression test, the cylinders were also tested for the static modulus of elasticity (ASTM C 469) and for dry unit weight. The dry unit weight was obtained by dividing the weight of the specimen by the measured volume of the specimen.

Static Flexure Test

The beams were tested for static flexural strength (ASTM C 1018) at ages 7 and 28 days. According to ASTM C 1018, the beams were tested over a simply supported span of 300 mm (12 inch) and third point loading was applied to the beams. The deflection was measured at the mid-span by using a dial gage accurate to 0.00254 mm (0.0001 inch). The deflections were measured using a specially fabricated frame. It was possible to measure the actual deflections eliminating all extraneous deflections due to the crushing of concrete and testing machine deformations. This test was a deflection controlled test. The rate of deflection was kept in the range of 0.05 mm to 0.10 mm (0.002 to 0.004 inch) per minute as per ASTM C 1018. The loads were recorded at every 0.00254 mm (0.0001 inch) increment in deflection until the first crack appeared after which the loads were recorded at regular intervals. The load corresponding to first crack and the maximum load reached were noted for each specimen. From the test results, load-deflection curves were drawn and ASTM toughness indices were calculated. The flexural toughness factor and equivalent flexural strength were also calculated using the Japanese standard method.

Impact Test

The specimens were tested for impact strength at an age of 28 days by the drop weight test method (ACI Committee 544). In this method, the equipment consisted of a standard manually operated 4.54 kg (10 lbs) weight with a 457 mm (18 inch) drop (compactor), a 63.5 mm (2-1/2 inch) diameter hardened steel ball, a flat steel base plate with a positioning bracket and four positioning lugs. The specimen was placed on the base plate with its rough surface facing upwards. The hard steel ball was placed on the top of the specimen and within the four positioning brackets. The compactor was placed with its base on the steel ball. The test was performed on a flat rigid surface to minimize the energy losses. The hammer was dropped consecutively, and the number of blows required to cause the first visible crack on the specimens was recorded. The impact resistance of the specimen to ultimate failure was also recorded by the number of blows required to open the crack sufficiently so that the pieces of specimen were touching at least three of the four positioning lugs on the base plate.

Flexural Fatigue Test

In this investigation, the determination of the fatigue strength was quite important as it is one of the main improvements in concrete due to addition of fibers. Fatigue strength is defined as the maximum stress at which the specimen withstood more than 2 million cycles of non-reversed fatigue loading. For most of the fiber reinforced concrete structures such as airport runways, highway pavements, and bridge decks, the 2 million cycles may represent typical fatigue loading over their life span.

Similar to the static flexural test, third point loading is used with a span of 300 mm (12 inch) on 100 mm x 100 mm x 350 mm (4 in. x 4 in. x 14 in.) beams. The lower limit for the dynamic loading was set at 10% of the average maximum loads from the static flexural test for the same mix category. The upper limit varied from 85% to 50% of the maximum load. If the beam failed before reaching 2 million cycles, then the upper load limit for the next beam was set at a lower percentage. If the beam survived 2 million cycles, then two more beams were tested at the same percentage.

The frequency of loading used was 20 cycles per second (Hz) for all fatigue tests. The MTS machine was used for both static and flexural fatigue tests. It has a cyclic load capacity of 25,000 kg (55,000 pounds). The control and monitor system consists of MTS 436 control unit, a Hewlett-Packard oscilloscope and a digital multimeter with an MTS load cell. The machine could be operated in any one of the three modes: Load control (force applied to the specimen), strain control (strain induced in the specimen) or deflection control (distance traveled by the ram or deflection of the specimen). Since this test was concerned with stress levels, load control was used for fatigue testing.

There was a choice of three wave forms that could be used: sine wave, square wave and triangular wave. In these experiments sine wave was used since it closely related to the actual cyclic loading behavior.

A counter was provided to keep track of the number of cycles to the nearest hundred. When the beam failed, the counter reading was recorded and multiplied by 100 to give the number of cycles the beam had been subjected to. A mechanical cut-off switch was provided which could turn off the machine when the beam failed.

Modulus of Rupture Test (Static Flexural Strength)

For fatigue investigation, the beams were tested for static flexural strength (modulus of rupture) according to ASTM C 78 which was a load-control test. The same size beam, 100 mm x 100 mm x 350 mm (4 x 4 x 14 -inch) was used with the same third point loading over a span of 300 mm (12 inches). The specimens were tested at various ages just prior to the commencement of the fatigue testing.

Static Flexure Test After Fatigue

The beams which survived more than 2 million cycles of non-reversed flexural fatigue loading, were tested again in static flexure, using the same procedure as described above (ASTM C 78).

Test Specimens

A number of test specimens were cast from all the mixtures. The following specimens were cast from each mix: 150 mm x 300 mm (6 in. x 12 in.) cylinders for compressive strength and static modulus tests, 100 mm x 100 mm x 350 mm (4 in. x 4 in. x 14 in.) beams for flexural strength, toughness tests and fatigue strength, 150 mm x 65 mm (6 in. x 2-1/2 in.) discs for impact strength.

All the steel, plastic and wooden molds were taken to the job site on the day prior to construction. The molds were well oiled. A portion of the fresh concrete from the ready-mix truck was discharged into a wheelbarrow to carry out the fresh concrete tests and to make specimens. The specimens were covered with plastic sheets and remained at the job-site for a period of 24 hours. They were then transported to the Concrete Technology Laboratory, SDSM&T, where they were demolded and placed in a lime saturated water tank for curing. The specimens remained in the curing tank until they were tested at the appropriate age.

Mixture and Specimen Designation

The following labeling procedure was used for all mixtures and specimens made from these mixtures:

The trial mixes for full depth pavement were conducted in the SDSM&T Concrete Laboratory on June 24, July 2, and July 12, 1996.

T3: Trial mix used for evaluating the performance characteristics of the concrete.

T4: Trial mix used for the Maturity testing of concrete.

T6: Trial mix used for the Fatigue testing of concrete.

Mix T3, T4, and T6 were respectively mixed on June 24, July 2, and July 12, 1996.

Full depth test sections were constructed on Highway 83, near northeast of Pierre, SD between MRM's 144 and 145. The sampled specimens collected from the paving were designated as follows:

P1: The North bound lane paved on August 15, 1996.

P2: The section of South bound lane paved on August 26, 1996.

P3: The remaining section of South bound lane paved on August 27, 1996.

PC: The North bound control section paved with plain concrete (no fibers), on August 15, 1996.

For cylinder specimens, the next character is C, for beam specimens, the next character is B, and for impact specimens, the next character is I.

Figures and Tables were also labeled as stated above.

The behavior of the pavement at joints and random cracks (uncontrolled cracks) were carefully recorded. The length and width of the random cracks were measured. The falling weight deflectometer (FWD) tests were conducted by the SDDOT and the results were analyzed and presented. Regular visual surveys were conducted to observe the behavior of the pavements at joints and random cracks. The visual inspections did not show any curling. Therefore no curling measurements were taken. Also curling probably would not appear in the pavements this soon after construction.

Periodic condition surveys were conducted to evaluate the performance of the constructed full depth pavement. These condition surveys were conducted at two weeks, one month, three months, six months, one year and at the latest possible date prior to the end of the contract.

Research Task 4: In consideration of NMFRC's enhanced properties, analyze the pavement designs proposed in tasks 5 and 6, assess their feasibility, and recommend design details such as joint spacing and initial saw cut depths for transverse and longitudinal joints.

In the actual construction 1) Two joint spacings 7.6 and 10.7 m (25 and 35 foot) were used in the NMFRC; 2) 393m(1290ft) of the NMFRC was unjointed; 3) Some of the NMFRC sections were doweled and others were not; and 4) Two NMFRC thicknesses were used. The reasons for using the above design criteria were as follows. For item number 1 above, SDDOT wanted to reduce joint maintenance, so with longer joint spacings (fewer joints) we would assume that there would be less maintenance. However spacings longer than these were not used because panel curling would then become a problem which would cause problems with the ride quality for the pavement. With longer panel sizes the movements at each joint would increase which might increase maintanence. Another concern was that the load transfer would be reduced if the joint spacings were increased too much. This was verified by FWD.

For item number 2 above, SDDOT wanted to determine what the crack pattern would be for continuos NMFRC pavements. Once the crack interval was determined, load transfer could be evaluated across the random cracks. The unjointed NMFRC pavement had random cracks at approximately 26m(85ft) intervals. The performance of the pavement at the random cracks was satisfactory. There were no spalling, ravellings, fatigue cracks or D-cracking at these random cracks. The riding quality, as assessed by observation, was the same as other jointed and control pavement sections. However the load transfer as indicated by FWD data (if it were assumed correct) was less across the random cracks. It should be noted that the number of fibers crossing the random cracks were reduced due to the routing and sealing operation which occurred after the cracks formed. Therefore, the ability of the fibers to transfer load across random cracks was reduced. For item number 3 above, SDDOT wanted to determine whether the fibers which spanned any random crack or control joint would affect the load transfer from one panel to the next. It was believed that the load transfer might be increased due to improved aggregate interlock resulting from the fibers ability to hold the random cracks and control joints tighter. This objective was not fully achieved because in order to determine how the fibers affect load transfer, the testing should have included doweled and undoweled 6.1m(20ft) NMFRC test sections such that they could have been compared to the doweled PJCP. FWD data indicated that the load transfer was less when the joint spacings were longer.

For item number 4 above, SDDOT wanted to determine how a thinner section would perform relative to the thicker NMFRC and control sections. It was believed that the improved properties of NMFRC would allow a thinner section to be used while maintaining the same performance. In addition, due to increase in initial cost resulting from the addition of fibers, it was believed that a thinner section would be necessary to offset some of the initial cost.

There is no theoretical or rational design procedure available to determine the exact joint spacings. Currently, the design is mainly based on past experience and performance and / or based on empirical equations. Literature review had shown that attempts were made to develop theoretical equations to determine joint spacing using finite elements and assumed elastic behavior for concrete. Since concrete is not an elastic material, a number of unreliable assumptions and empirical constants had been used in the derivation of the equations. There was no method of determining the joint spacings for NMFRC suggested in the literature.

The joint spacings used for the control section was 6.1m(20ft) whereas 7.6 and 10.7m(25 and 35 ft) joint spacings were used for the NMFRC. The performance of the NMFRC joints was satisfactory. However the load transfer as indicated by the FWD data was less in pavement with longer joint spacings.

Based on the observed short term performance of the joints in the NMFRC, it is recommended that longer joint spacings could be used for NMFRC pavement. For thicker full depth NMFRC pavements (203mm (8.0in)) a 15.25m(50 ft) joint spacing is suggested, and for a thinner pavement (165mm (6.5in)) 10.7m (35ft) joint spacing is recommended. This recommendation is based on the assumption that the FWD data did not truly reflect the load transfer in NMFRC pavements.

The initial saw cut depths used for transverse and longitudinal joints were the same as traditionally used by SDDOT. The same saw cut depths were used for control and NMFRC sections. The saw depths used for transverse joints and longitudinal joints were T/4 and T/3 respectively where T is the pavement thickness. The standard pavement dowel bar assembly for transverse contraction joints was used whenever dowel bars were needed. The performance of transverse and longitudinal joints was satisfactory. There were no joint damages. Since different saw depths were not used in this project, a comparative evaluation was not possible. Therefore it is recommended that SDDOT's standard saw depths used for transverse and longitudinal joints could be used for NMFRC. However undoweled joints are not recommended because the load transfer is less across undoweled joints. Two thicknesses, 203mm(8 in) and 165mm(6.5 in) were used for the NMFRC full depth pavements. The performance of these two thicknesses in regard to riding quality was the same. There was no damage of any type in these two pavement sections. There was no fatigue cracking, curling, a faulting in either section. However the FWD data had shown that the load transfer was less in the thinner section pavement, which was anticipated. However the correctness of this data has yet to be verified. Therefore it is suggested that thinner sections (165mm (6.5in)) could be used for NMFRC pavements.

Sub-Contract:

In order to obtain design criteria to determine the pavement thickness, and joint spacing for NMFRC pavements, a sub contractor, Dr. Shiraz D.Tayabzi Ph.D., P.E, regional vicē president of ERES consultants, Inc; was hired. According to the consultant agreement signed on July 26,1996 by Dr. Shiraz Tayabji, the consultant work would be performed by Dr. Shiraz Tayabji and would include providing the thickness of the pavement for the NMFRC using concrete pavement thickness design procedures and providing the maximum joint spacing for the full depth pavement using the NMFRC. A written report would be provided by August 20, 1996 detailing the calculations and the equations and procedures used. All the information including the properties of the NMFRC and the anticipated traffic details was supplied to him immediately.

The subcontractor did not give the design. A number of reminders were sent and no reply was received. Whenever contacted by phone, he would say that he would give the information within a week, which was never done. Finally a meeting was held in Washington D.C in Jan 1998 during the TRB Annual meeting in which the sub contractor Dr. Tayabji, the P.I, and the SDDOT's Office of Research Program Manager. Dr. Tayabji again agreed to give the design in a week. The design and agreed details were never supplied by the sub-contractor.

The P.I contacted another expert who had done SFRC pavement designs. He informed the P.I that there was no separate design procedure available for FRC and the procedure used by us was the procedure he used in his designs.

Research Task 5: Evaluate NMFRC full depth pavement test sections from design through construction and subsequent service performance with special attention to pavement distress and load transfer. The panel envisions the following test and control sections: two doweled 152m (500ft.) test sections each with a different joint spacing; two undoweled 152m (500ft.) test sections each with a different joint spacing; one unjointed 390m (1280ft.) test section; and one 152m (500ft.) control section. Thickness will be identical to the rest of the paving project.

a) In conjunction with SDDOT design personnel, review the design and plans developed for the pavement construction.

d) Attend preconstruction meeting(s) and recommend NMFRC construction methods.

The P.I. attended a facilitated pre-paving meeting organized by the Contractor on July 16, 1996 in Pierre, SD. The P.I. also attended the Preconstruction meeting on the same day. The preconstruction meeting was presided by Norman Konechne, Pierre Area Engineer. The schedule for construction was presented during this meeting. Issues like the procedure for fiber addition and joint cutting were also discussed during this meeting.

It was decided that DOT would conduct the fresh concrete's quality control tests (acceptance testing), however, the research team would take additional samples for purposes of the research. The project also consisted of the following: conducting fresh concrete tests during the construction, analyzing and evaluating the test sections, and making recommendations. An extensive laboratory investigation would be conducted to recommend the appropriate mixture proportions for the construction of the full depth pavement, and to fully determine the fresh and hardened concrete properties of these recommended concretes.

Throughout the project, the work was done in consultation and with approval from the Technical Panel.

Open House:

An Open House was organized by the South Dakota Chapter, American Concrete Pavement Association July 16, 1996 in Pierre, SD. The P.I. participated in the Open House and discussed the project details. Another Open House was organized by SDDOT, 3M Company, American Concrete Paving Association, SD School of Mines and Technology, and Federal Highway Administration. This Open House was on October 15 and 16, 1996, in Pierre, SD. The P.I. presented a lecture on "Fiber Types and Differences".

Research Task 5b: In conjunction with SDDOT design personnel, design the concrete mix.

A mixture proportion was selected based on previous laboratory and field experience in the construction of NMFRC full depth pavement prior to the start of the project. Actual aggregates to be used in the project were obtained from the contractor and trial mixes were made at the South Dakota School of Mines and Technology Concrete Laboratory. Based on the trial mixes and in consultation with the Technical Panel, the final mixture proportions were selected.

The following mixture proportions were recommended for the Full depth pavement concrete:

Cement (Type II)	302 kg/cu m (510 lbs./cu. yd.)
Fly Ash	66 kg/cu m (112 lbs./cu. yd.)
(Lafarge Coal Creek Plant-Modified "F")	
Water	157 kg/cu m (264 lbs./cu. yd.)
Lime Stone Coarse Aggregate	841 kg/cu m (1417 lbs./cu. yd.)
(1990 SDDOT Spec. Size 1)	
Fine Aggregate	841 kg/cu m (1417 lbs./cu. yd.)
Polyolefin Fibers	14.8 kg/cu m (25 lbs./cu. yd.)
Slump	25 to 50 mm (1 to 2 inches)
Air Content	6 ± 1.5 %

Research Task 5c: Conduct tests on the mix design(s) hardened concrete to ensure desired properties are obtained

The batches for full depth pavement were conducted in the SDSM&T Concrete Laboratory on June 24, July 2, and July 12, 1996. The batch designated as T3 was used for evaluating the performance characteristics of the concrete. The batch designated as T4 was used for the Maturity testing and batch T6 for the Fatigue testing of concrete. Batch T3 was mixed on June 24, batch T4 was mixed on July 2, and batch T6 on July 12, 1996. The performance characteristics of the batches are included in the Appendix A. The Maturity testing was done as per ASTM C1074-93. For the Maturity testing, two cylinders were tested in compression for one, three, four, five, seven, fourteen, and twenty eight days. Maturity meter readings were taken from two cylinders from the same batch with each cylinder containing one probe. Details of the maturity testing are also included in Appendix E.

The proportions used in the trial mixes gave a good workable concrete and the fibers were well distributed in the concrete. The fibers mixed well without causing any balling, segregation, or fiber lumping.

The slump can vary depending on the air content, the concrete temperature, ambient temperature and wind conditions. Medium or High Range Water Reducers

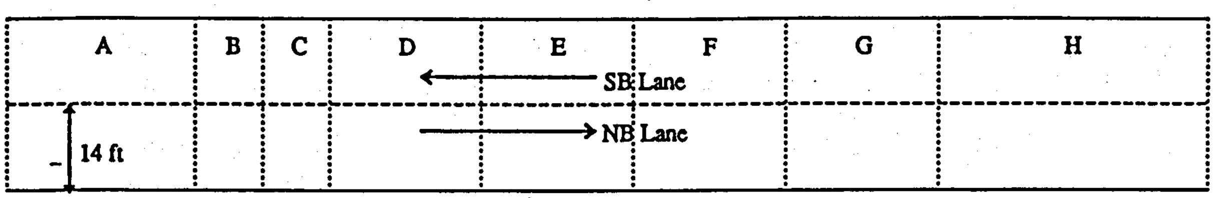
(Superplasticizers) may be added at the plant or at the job-site to increase the slump to the specified range.

The quantity of air entraining agent will depend on many factors such as type of air entraining agent, the concrete temperature, slump, and ambient weather conditions. The contractor was to use the appropriate amount of air entraining agent.

Research Task 5e: Perform quality control testing, record weather conditions, and observe and record construction activities.

Full depth NMFRC and Control test sections were constructed on US 83 northeast of Pierre, South Dakota between mileage reference markers (MRM's) 144 & 145. Two lanes, each 14 ft wide of the following test sections as shown in the figure and table below were constructed:

Pavement Test Section Layout (Near MRM 145)



Test Section	Length (ft)	Type	Thickness (in)	Doweled	Joint Spacing (ft)	Volume of Fiber Concrete (yd³)
A	1000	Plain PCCP Control	8	Yes	20	0
B	250	Fiber (25 lbs/yd³)	6.5	No	25	173
C	245	Fiber (25 lbs/yd³)	6.5	No	35	170
D	500	Fiber (25 lbs/yd³)	8	Yes	25	346
E	490	Fiber (25 lbs/yd³)	8	Yes	35	339
F	500	Fiber (25 lbs/yd³)	8	No	25	346
G	490	Fiber (25 lbs/yd³)	8	No	35	339
H	1290	Fiber (25 lbs/yd³)	8	No	None	892
					Total	2,605

1in = 25.4mm 1ft. = 0.305m 1 yd^3 = 0.765 m^3 1 lb/yd^3= 0.593 kg/m^3

The North Bound lane of the test section was paved on August 15, 1996. The paving started around 7:30 AM. The research team was on the site from 5:30 AM, to set up the equipment and to make preparations for the concrete testing. The first sample was taken at 7:45 AM. The test section was completed around 7:30 PM. The ambient temperature varied from 12.8°C to 33.3°C (55°F to 92°F) and the humidity varied from 80 % to 30 %. At the beginning of the paving the wind velocity was 0-2 mph. By noon the wind velocity was 14 mph. The concrete temperature varied from 26°C to 31.3°C (78.9°F to 88.4°F). The research team randomly sampled the concrete every hour and tested it for fresh concrete properties. Specimens were made from the sample concretes.

The South Bound lane was paved on August 26 and 27, 1996. The paving of the test section started around 3:00 PM. The research team was on the job-site from 11:00 AM to set up the equipment and to make preparations for the concrete testing. The first sample was taken at 3:05 PM. Construction continued till 7:30 PM. The ambient temperature varied from 22.7°C to 30°C (73°F to 86°F) and the humidity varied from 35% to 20%. At the beginning of the paving the wind velocity was around 7 mph. By evening the wind velocity was 14 mph. The concrete temperature varied from 27.6°C to 28.7°C (81.7°F to 83.6°F). The research team randomly sampled the concrete every hour and tested it for fresh concrete properties. Specimens were made from the sample concretes.

The remaining portion of the South Bound lane of the test section was paved on August 27, 1996. The paving of the test section started around 7:30 AM. The research team was on the job-site from 6:30 AM to set up the equipment and to make preparations for the concrete testing. The first sample was taken at 7:55 AM. Construction continued till 11:00 AM. The ambient temperature varied from 10°C to 22.8°C (50°F to 73°F) and the humidity varied from 75% to 58%. At the beginning of the paving the wind velocity was around 0-2 mph. By late morning the wind velocity was 9 mph. The concrete temperature varied from 23.4°C to 25.1°C (74.2°F to 77.2°F). The P.I., R.A., two graduate students, and one undergraduate student attended the construction of the full depth pavement. Concrete was sampled randomly at every hour and tested for fresh concrete properties. Specimens were made from the sample concretes.

Construction Procedure:

Prior to construction of the NMFRC test sections, two items of concern had to be addressed. These items were: 1) How and where should the fibers be added into the batching process? 2) Will the fibers be evenly distributed in the mix when SDDOT's specified minimum mixing time is 1 minute for concrete batch plants?

Several tests were conducted which addressed item number 1 above. From the testing it was found that the fibers could be added to the batching process by introducing them into an aggregate bin through the opening between the store and weigh bins. The fibers could be added to the aggregate bin by mounting a large diameter plastic pipe such that one end of it was in the aggregate bin. The pipe was long enough to hold 7 boxes of fibers (1 box/yd^3 of concrete). The pipe had a trap door at the end nearest the aggregate bin and was mounted at an angle such that when the trap door was removed, the fibers would be introduced into the bin. Sand would then be added to the bin on top of the fibers. Then the bin's door would open, the fiber bundles and sand would drop onto the conveyor and conveyed into the mixing drum. By adding the fibers to the sand bin, the sand was able to hold the fiber bundles on the conveyor and prevented them from rolling back down the belt.

Testing was also done to address item number 2 above. Because SDDOT's earlier projects, each used NMFRC delivered by mixing trucks, the mixing process was quite different and allowed a much longer (5 to 6 minutes over and above that needed to mix the concrete) mixing time as compared to the 1 minute minimum required for batch plant concrete. Once all the ingredients were in the drum, the 1 minute timer would begin. Because of this it was obvious that the fibers should be added as early in the batching process as possible. This would allow them to be mixed for a longer period of time. In addition, while the bundles were in the plastic tube, the fiber bundles were pre-wet which allowed the paper to soften prior to the bundles' addition into the mixing drum. It should be noted that the water used to pre-wet the bundles added about 2lbs. of water per cubic yard of concrete.

After addressing these concerns it was found that an average of 80 seconds to mix the concrete was needed to evenly distribute the fibers. Fortunately, for this construction project, the slight increase in mixing time did not slow the paving process.

The contractor and concrete supplier was, Stanley J. Johnsen, Concrete Contractor Inc. Fibers were mixed with concrete at the central batching plant and then the NMFRC was supplied in dump trucks. The paving was done by a concrete paver (Auto Float Gomaco Paver). The NMFRC was as workable, placeable, and finishable as plain concrete. The curing specifications and procedures were followed as per SD DOT standards.

For a typical day, the following construction procedures using fiber concrete were observed. The concrete was mixed at the central batching plant 5.35 cu. m. (7 cu. yd.) at a time. By opening the trap door in the plastic pipe, the fiber bundles were discharged from the tube into the sand bin. Next, the sand, coarse aggregates, and fibers were conveyed into the mixing drum. Then the cement and water were added and mixed for about 80 seconds in the drum rotating at mixing speed. The whole mixing operation - adding materials and mixing, took about 1 minute 45 seconds to 2 minutes. The concrete was discharged into an end-dump dump truck which delivered the concrete to the spreader. The paving machine followed the spreader. The paver vibrators consolidated the concrete. A wet burlap was dragged by the paver on the consolidated concrete behind the paver. Then an automatic finishing float ("Auto Float") which was attached to the paver finished the concrete. Two persons were finishing the pavement's edges and then a bull float was used. After bull floating, Astro-turf dragging was done in the direction of the traffic. Then an automatic tining machine did the tining in the transverse direction. Then followed the curing compound spraying which was also done by a machine.

Since each lane was paved separately, tie bars were inserted every 1 m (3 ft) center to center along the centerline. The paver itself automatically inserted the tie bars. A person was placing the dowel bars on the machine.

Rumble strips were installed followed by the tining operation. The joints were cut later at the discretion of the contractor. The P. I. discussed the timing of the saw cutting of the joints with the contractor.

All the machines used were Gomaco make: Auto Float Gomaco Paver - GHP 2800, Gomaco Spreader PS 48, Gomaco Turf Drag C 450, Gomaco Tining Machine, and Gomaco Automatic Curing Compound Sprayer.

The number of additional people needed at the plant to add the fibers were: two persons opened the boxes and loaded them on the conveyor, one person operated the conveyor, one to dump the boxes into the tube, one to open the trap door, and one to prewet the bundles in the pipe so that they could quickly open in the drum. These were the six extra persons needed for the fiber concrete paving compared to the plain concrete paving.

Quality Control Tests for Fresh Concrete

For both lanes, fresh concrete was tested for slump (ASTM C 143), air content (ASTM C 231), and fresh concrete unit weight (ASTM C 138). The temperature of the fresh concrete was also recorded.

The properties of the fresh concrete used in the full depth pavement are given in Tables B1 and B8. The slumps and air content measured were satisfactory and they were almost within the range specified by the DOT. The unit weights calculated did not vary much.

The actual measured fiber contents in the samples taken from the field concrete were close to the specified amounts (except in the south bound lane at section around station 317±10 placed on August 26, 1996) and are recorded in the table of fresh concrete properties. (Table B8)

When observing the delivered concrete, it was noticed that there were not adequate fibers in one dump truck. Therefore this concrete was tested for fiber content. This might have happened due to one of the following reasons: 1) The person at the plant might not have opened the trap door of the plastic tube and hence no fibers were added in that particular drum. 2) A smaller number of boxes were loaded into the plastic tube instead of the required number of boxes for this particular drum.

The measured slumps are shown in Figure B1 page B-23 and the air contents are given in Figure B2 page B-24. The fresh concrete unit weights are given in Tables B1 and B8 pages B-2 and B-7.

The number of specimens cast during each of the quality control sampling are given in Tables B2 and B9.

Research Task 5f: Conduct hardened concrete performance tests on the collected field samples.

Quality Control tests for Hardened Concrete Properties

The compressive strengths and elastic modulus values are given in Tables B3 and B10 pages B-3 and B-8. A comparison of the compressive strengths results for different batches are shown in Figure B3 page B-25. The 7 day average compressive strengths recorded were 26.6 MPa (3865 psi for P1), 24.1 MPa (3500 psi for P2), and 25.6 MPa (3715 psi for P3). The 28 day average compressive strengths recorded were 31.2 MPa (4530 psi for P1), 28.1 MPa (4075 psi for P2), 34.9 MPa (5055 psi for P3), and 30.4 MPa (4415 psi for PC), which was a tolerable variation in the field concrete. The specified compressive strength was 27.6 Mpa (4000 psi). The variation in the elastic modulus values was consistent with that of the compressive strength variation.

The first crack strength and the modulus of rupture values are given in Tables B4 and B11 pages B-4 and B-9 and Figures B4 and B5 pages B-26 and B-27. There was no significant variation in the modulus of rupture for different batches.

The toughness indices, calculated according to the ASTM standard procedures, are given in Tables B6 and B13 pages B-5 and B-6. The first crack toughness is compared in Figure B6 page B-28 and the toughness indices I5, I10, and I20 are compared in Figures B7 and B8 pages B-29 and B-30. The ratios of I10/I5 and I20/I10 are compared in Figures B9 and B10 pages B-31 and B-32.

The ASTM toughness indices were approximately the same at 7 and 28 days which was normally expected. Toughness indices I5, I10 and I20 were respectively 4, 8, and 15 times higher than that of plain concrete because for plain concrete these values were 1. The ratios I10/I5 and I20/I10 indicated a very ductile behavior.

The Japanese standard toughness and equivalent flexural strengths are shown in Tables B5 and B12 pages B-5 and B-10 and Figures B11 page B-33 and B-34 page B-6, respectively. The comparisons also confirmed the increase in ductility and toughness of the concrete due to the addition of polyolefin fibers.

The impact resistance of the concretes are given in Table B7 page B-6. There was a high impact strength due to the addition of polyolefin fibers in the concrete. The number of blows for ultimate failure in NMFRC was above 200. The impact test for plain concrete was not done. But based on the knowledge of the previous research experiments it would be between 20 and 30.

Flexural Fatigue Strength and Endurance Limits for Plain and FRC

One of the most important contributions due to the addition of the fibers in the concrete is the change in the mode of failure of concrete when subjected to fatigue loading. The addition of fibers converts a sudden and brittle failure to a more desirable ductile failure. There is also an increase in fatigue strength and endurance limit when fibers are mixed with concrete. As a part of this project an extensive investigation was conducted to evaluate the performance of NMFRC subjected to fatigue loading and compare its performance to that of plain concrete. Specimens from the following concrete were tested and analysed. 1) Trial batches made in the laboratory (DOT-5 and DOT-6), 2) actual concrete used for the pavement construction (P1), and 3) the actual concrete (W1) used for construction of Whitetopping in Highway 14 (Study SD 96-13).

Notation

f'_c - Compressive strength of concrete

f_r - Average static flexural stress

f_{max} - Flexural fatigue stress (Maximum)

f_{rn} - Normalized average static flexural stress ($f_r \times \sqrt{5500}/\sqrt{f_c}$)

$f_{max,n}$ - Normalized flexural fatigue stress (Maximum) ($f_{max} \times \sqrt{5500}/\sqrt{f_c}$)

f_{rA} - Static flexural strength of beams after it had been subjected to fatigue loading

E.L. - Endurance Limit

N.E.L - Normalized Endurance Limit

In many applications, particularly in pavements and bridge deck overlays, the flexural fatigue strength and endurance limit are important design parameters because these structures are designed on the basis of fatigue load cycles. The greatest advantage of adding fibers to concrete is the improvement in fatigue resistance. A properly designed FRC can achieve a 90 to 95 percent endurance limit. Theoretically, with a higher endurance limit, the concrete cross-sections could be reduced. Alternatively, using the same cross section could result in a longer life span or higher load carrying capacity or both.

There was a need to determine the fatigue strength of newly developed materials such as NMFRC.

Flexural Fatigue Strength of FRC

In recent years, considerable interest had developed in the flexural fatigue strength of concrete members. The widespread adoption of ultimate strength design, and use of higher strength materials required that structural concrete members perform satisfactorily under high stress levels subjected to a large number of load cycles. In many structural applications (like pavements, bridge deck overlays, crane beams, and offshore structures) the flexural fatigue and endurance limit are needed properties.

Hardened Concrete Properties

Additional beams and cylinders were made from NMFRC (Mixes T5, T6, P1 and W1) and tested to determine the hardened concrete properties and fatigue performance.

The compressive strength, static modulus values and ages of testing are reported in Table B14 & B15. The average compressive strength and static modulus values are compared in Figures B13 & B14. The average compressive strengths for mixes T5, T6, P1, W1 and plain concrete were 49.89 MPa (7235 psi), 52.22 MPa (7574 psi), 33.54 MPa (4865 psi), 31.77 MPa (4608 psi) and 42.96 MPa (6230 psi) respectively.

Prior to the commencement of fatigue testing, three randomly selected beams were first tested for modulus of rupture according to ASTM C 78. The average of these results was considered as the static flexural strength (f_r). The modulus of rupture results are reported in Table B16 & B17. The average static flexural strength values of plain concrete and the NMFRC mixes are compared in Figure B15. The average static flexural strength (f_r), for mixes T5, T6, P1, W1 and plain concrete were 6.16 MPa (892 psi), 6.93 MPa (1004 psi), 5.89 MPa (854 psi), 6.55 MPa (949 psi) and 5.10 MPa (739 psi) respectively.

Flexural Fatigue Test

Flexural fatigue strength (f_{max}) was defined as the maximum flexural stress for which the specimen could withstand more than 2 million cycles of non-reversed fatigue loading. For reinforced concrete structures such as airport runways, highway pavements, and bridge decks, the 2 million cycles might represent the typical fatigue loading over their life span. In the test for flexural fatigue, the same test set-up as for the static flexural strength was used (third point loading over a span of 305 mm (12 inches)). The lower load limit was set at 10 percent of the average f_r and the upper load limit (f_f) was set between 50 to 90 percent of f_r. If the beam failed before reaching 2 million cycles, then the upper limit for the next beam was set at a lower percentage. If the beam survived two million cycles, then two more beams were tested at the same percentage. The frequency of loading used was approximately 20 cycles per second for all tests. The same MTS machine was used for both static and flexural fatigue tests.

The fatigue test results including the dimensions of the specimen, the maximum load, the number of cycles to failure, the fatigue stress (f_f) for this particular loading, and the ratio f_f/f_r are reported in Tables B19 to B22. The fatigue strengths (f_{max}) were obtained by drawing figures of the flexural fatigue stress f_f/f_r vs. number of cycles and f_f/f_r vs. log number of cycles as shown in Figures B16 to B24.

The calculated flexural fatigue strengths (f_{max}) are reported in Table B18. The f_{max} values for mixes T5, T6, P1, W1 and plain concrete were 3.49 MPa (506 psi), 4.18 MPa (606 psi), 4.15 MPa (602 psi), 4.21 MPa (611 psi) and 3.45 MPa (501 psi) respectively. The flexural fatigue strength of mixes T6, P1 and W1 was higher than that of plain concrete by 21%, 20% and 22%, respectively. The flexural fatigue strengths (f_{max}) of all the mixes are compared in Figure B25.

Normalization

There was a variation in the compressive strengths of the NMFRC mixes compared above. In order to compare the flexural strength, such as static flexural strength (f_r), flexural fatigue strength (f_{max}), and endurance limit (E.L.) for all concretes on an equal compressive strength basis, a normalization procedure was used. It was well established in literature and codes that the flexural strength of concrete varied proportionally to the square root of the compressive strength of the concrete (ACI Building Code 318). Therefore, all comparisons were made for a nominal compressive strength of 38 MPa (5500 psi). For calculating the normalized static flexural strength (f_{rn}) and normalized flexural fatigue strength ($f_{max,n}$), the following equations were used.

$$f_{rn} = f_r \sqrt{5500} / \sqrt{f_c'}$$

$$f_{max,n} = f_{max} \sqrt{5500} / \sqrt{f_c'}$$

The normalized values calculated are reported in Table B18. It was clearly indicated that the addition of polyolefin fibers had considerably increased the fatigue strength of concrete. The normalized fatigue strengths ($f_{max,n}$) were 3.25 MPa (471 psi), 3.04 MPa (441 psi), 3.56 MPa (516 psi), 4.41 MPa (640 psi) and 4.60 MPa (668 psi), respectively for plain concrete, T5, T6, P1 and W1. The mixture W1 had the highest increase of 42% with respect to plain concrete. Mixes P1 and T6 had 10% and 36% increase, respectively. The normalized flexural fatigue strengths, $f_{max,n}$ are reported in Table B18.

The flexural fatigue strength of T5 and plain concrete were almost the same. The reason for this was not clear. It might have been that a correct calibration chart was not used in recording the loads during the testing of T5. The testing machine had been repaired for leaks and the testing of T5 was done after the repair.

In general the fatigue strength of NMFRC was not as high as found in earlier work reported in SDDOT 94-04 study. This might have been due to a lower bond development between the fibers and the concrete.

Fatigue Life Prediction

The S-N curves were used to predict the fatigue life for concrete. The curves depicting the ratio of flexural fatigue strength to static flexural strength (f_f/f_r) vs. the number of cycles and (f_f/f_r) vs. the log number of cycles, for all mixes are plotted in Fig. B16 to B24. There was a linear relationship between the flexural fatigue stress (f_f) and the log number of cycles, until the fatigue strength (f_{max}) of that particular concrete was reached then the line became parallel to the x-axis indicating that there was an endurance limit for NMFRC. The same type of behavior was observed for all mixes. There was an increase in the fatigue strengths and fatigue life with the addition of polyolefin fibers compared to plain concrete. W1 has a higher increase in fatigue life compared to other mixtures.

Endurance Limit based on its own Modulus of Rupture

The Endurance Limit, EL_1 could be defined as the flexural fatigue stress at which the beam could withstand 2 million cycles of non-reversed fatigue loading (f_{max}) expressed as a percentage of the modulus of rupture (f_r). Thus defined EL_1 is compared for plain concrete and NMFRC mixes in Figure B26 . These values are also reported in Table B18. The endurance limits, EL_1 vs. number of cycles and EL_1 vs. log number of cycles for the four NMFRC mixes are shown in Figures B16 to B24.

Endurance Limit based on Plain Concrete

The endurance limit, EL_2 was defined as the maximum flexural fatigue stress at which the beam could withstand 2 million cycles of non-reversed loading, expressed as a percentage of modulus of rupture of plain concrete. The EL_2 values are reported in Table B18. The comparison of endurance limits is shown in Figure B27. Mixes DOT-T6, P1 and W1 had higher endurance limits compared to plain concrete. DOT-T6, P1 and W1 had 20%, 19% and 22% higher endurance limits respectively.

Normalized Endurance Limit

Due to variation in the compressive strengths of the NMFRC mixes, a normalization procedure was used to compare the endurance limit for all concretes on an equal compressive strength basis. The normalized endurance limit (NEL_2), based on plain concrete is defined as the ratio of the normalized flexural fatigue strength to the normalized static flexural strength of plain concrete ($f_{max,n}/f_{rnc}$), expressed as a percentage. The normalized endurance limits are also reported in Table B18. Mixes DOT-T6, P1 and W1 had higher endurance limits compared to plain concrete. Mixes DOT-T6, P1 and W1 had 9%, 35% and 41% higher endurance limits (NEL_2), respectively. The normalized endurance limits of all the mixes are compared in Figure B28.

Residual Static Flexural Strength After Fatigue Loading

Beams that had withstood 2 million cycles were further tested for static flexural strength, f_{rA}. These results are reported in Table B23 & B24. There was an increase in the static flexural strength for mixes T5, T6, P1 and plain concrete. The observed increase was higher than the strength gain that could be attributed to the aging of the test specimens. Mixes T5, T6 and P1 had 27%, 12% and 28% increase in the static flexural strength respectively. The comparison of static flexural strength for all mixes, before and after fatigue_testing is shown in Fig. B29. The amount of increase in the static flexural strength seemed to depend on the flexural fatigue stress (f_f) to which the specimens were subjected to during the fatigue testing. With lower f_f values, the increase in f_r was higher. It was suggested that this post fatigue increase in f_r was due to densification of concrete caused by initial low stress level cycling, in a manner similar to the improvement in strength under moderate sustained loading.

Testing of Core Samples

Four core samples for the north bound lane and six core samples for the south bound lane were taken by the DOT personnel. The details about the visual inspection of the cores and its diameter and height are given in Tables D1 and D2 on pages D-2 and D-3. Before testing the cores, they were trimmed using a Hillquist saw. Its dimensions were again measured accurately using a Vernier. The cores were tested in Compression as per ASTM. Since the length of the cores was less than twice its diameter, a correction factor as per ASTM C42, was used for calculating the compressive strength. The details of the core testing are given in Tables D3 and D4 on page D-4 and D-5.

Note: The cores were taken to ensure that the proper concrete depth was attained.

Task 5g: Periodically conduct condition surveys to evaluate the field performance of the constructed pavement.

North Bound Lane:

The North Bound lane was paved on August 15, 1996. The first and second inspection of this lane was done on August 16 and 17, 1996. The P.I., R.A., two graduate students and one undergraduate student participated in the survey on the first day. The P.I. conducted the survey on the second day. One random crack was located on the first day and five more random cracks were located in the unjointed section of the pavement on the second day. Crack widths were measured at three places, two edges and center, on the surface of the pavement. In the jointed section, cracked joints were measured on both the edges. The depth of pavement at all the joints were accurately measured.

P.K. nails were placed across the joints in the jointed section by the DOT, immediately after the paving. The distance between the P.K. nails was recorded accurately after placing. The Research team measured the distance between the nails after one day. The P.K. nail distances were measured accurately to 1/16th of an inch.

The third inspection of the North Bound lane was done on August 25, 1996. Seven more random cracks were located in the unjointed section making the total number of random cracks equal to thirteen. The random cracks were measured at five locations.

Two on the vertical edges of the pavement and three on the surface of the pavement. The distance between the P.K. nails for all joints were also measured during this inspection.

The fourth inspection of the North bound lane was done on September 7, 1996. Only one new random crack was located in the unjointed section, making the total number of cracks equal to fourteen. The distances between the P.K. nails for all joints in the jointed sections were also measured during this inspection.

The fifth inspection of the North bound lane was done on October 16 and 17, 1996. Before the fifth inspection all the random cracks in the unjointed section were routed and sealed. On September 10, 1996, the DOT had placed P.K. nails across the random cracks, and measured the distance between the nails. During the fifth inspection, distances between the P.K. nails were measured for all the random cracks and joints for the North bound lane.

The sixth inspection of the North bound lane was done on May 28, 1997. Two new random cracks were located in the unjointed pavement. The crack widths and locations were noted. The seventh inspection was done on July 28, 1997. No new cracks were found during this inspection. The eighth and ninth inspections were done respectively on November 7, 1997 and April 17, 1998. No new cracks were found during both these inspections.

South Bound Lane:

The South Bound lane was paved on August 26 and 27, 1996. The first inspection of this lane was done on August 27, 1996. The P.I., R.A., one graduate student and one undergraduate student participated in the survey. Nine random cracks were located in the unjointed section of the pavement. These nine cracks were exactly in-line with nine cracks on the North bound lane. Crack widths were measured at four places along the length of the crack. In the jointed section, cracked joints were measured on the west vertical edge. The depth of pavement at all the joints were accurately measured.

P.K. nails were placed across the joints by the DOT, immediately after the paving. The distance between the P.K. nails were recorded accurately after placing. The

Research team measured the distance between the nails after one day. The P.K. nail distances were measured accurately to 1/16th of an inch.

The second inspection of the South Bound lane was done on September 7, 1996. Thirteen random cracks were located in the unjointed section of the south bound lane. All thirteen cracks were exactly in-line with the thirteen cracks on the North bound lane. The cracks were measured at three locations on the surface of the pavement. The distance between the P.K. nails for all joints were also measured during this inspection.

The third inspection of the south bound lane was done on October 16 and 17, 1996. Before the third inspection all the cracks in the unjointed section were routed and sealed. On September 10, 1996, DOT had placed P.K. nails across the cracks, and measured the distance between the nails. During the third inspection, distances between the P.K. nails were measured for all the random cracks and joints for the South bound lane.

The fourth inspection was done on May 28,1997. New random cracks were located in the unjointed NMFRC pavement. The crack widths and their locations were noted. The fifth inspection was done on July 28, 1997 and no new cracks were found. The sixth and seventh inspections were done respectively on November 7, 1997 and April 17, 1998. No new cracks were found during these inspections. The distances between the PK nails were measured during all these inspections.

The crack locations in the unjointed NMFRC pavement are shown in Figure C1 in Appendix C. These cracks formed at approximately 26m (85ft .) intervals and they were almost straight transverse cracks, which run continuously across both lanes. They appeared to be similar to the regular sawed joints.

The differences in the PK nail distances measured across the cracks in the unjointed pavement are given in Table C1 in Appendix C.

The distances between the PK nails for all the joints measured during all the inspections are given in Table C2.

There is no consistent pattern shown in the crack measurements and hence no positive conclusions are made based on these results. In some cases, the differences (representing the crack widths) increased, the maximum being 4.7mm (0.185in.). In some other cases, the crack widths decreased as much as 3.1mm (0.125in.). In some other cases there was no change, indicating no cracks.

An improvement is needed in the devices used for measuring the joint openings. The accuracy of the measurement was limited to 1.6mm (0.0625in.) and this was not adequate for a detailed analysis. There was also some movement in the nails, probably due to the traffic, and there was also raveling at the holes of the PK nails.

In all the inspections, no random cracks were found in the control section or the jointed NMFRC sections. There was also no significant joint spalling, raveling, cracking or pop-outs in either the control or NMFRC pavements.

Research Task 6: Design undoweled NMFRC full pavement test sections by modifying the PCA pavement thickness design method as outlined in SD95-20 and using appropriate NMFRC test data. Evaluate an NMFRC pavement from design through construction and subsequent service performance with special attention to the effects of pavement load transfer and joint spacing. The Panel envisions the following test sections: two 76m (250ft) test sections each with a different joint spacing. The control section used for comparison will be the same as that used in Task 5.

As stated in Task 4, a thinner section was selected because it was believed that the enhanced properties of the NMFRC would allow a thinner section to be used while maintaining the same performance as PJCP. In addition a thinner section would be more economical. Therefore two 76m(250ft) test sections with 7.6m(25ft) and 10.7m(35ft) joint spacings were constructed.

The literature survey had indicated that there was no design procedure available for NMFRC. In an earlier research project (SD94-04) a thinner section (140mm(5.5in)) of NMFRC pavement was successfully constructed.

The performance of that pavement was good. In designing that pavement, a modified PCA method was used by substituting the actual NMFRC properties such as fatigue strength and flexural strength instead of plain concrete properties. There was a 50% increase in the flexural fatigue strength of NMFRC as compared to plain concrete. Therefore a 30% reduction in the thickness would be possible. Therefore 30% of

SDDOT's 203mm(8in) minimum thickness would be 140mm(5.5in). However SDDOT did not feel comfortable with this thickness because of the likeliness of variability in the subgrade which would give a thinner section than 140mm(5.5in). Therefore it was decided that 165mm(6.5in) of NMFRC, computed from the AASHTO design equation would be tolerable.

Tests conducted by SDDOT.

Falling Weight Deflectometer Test

The falling weight deflectometer test was conducted by the SDDOT engineers and the summary results are given in Appendix F. A sketch of the test sections and the dimensions and other details of the test sections A to H are also given in the same page. Each joint or random crack was tested in each section. The results were summarized for each section.

In general the load transfer seems to be less in the NMFRC section (B to H) compared to the control section A. The lowest load transfer percent is 65.7 for the unjointed NMFRC test section H. The highest value 92.1% is for section D, which is NMFRC- 203 mm (8 in.) thick doweled pavement with 7.6m (25 ft) joint spacing. In general, the load transfer seems to be less in sections with longer joint spacings, smaller thickness 165mm (6.5 in.), and undoweled sections. The results were as anticipated. Sections with longer joint spacings would have wider cracks and hence the aggregate interlock effect may be less. Hence the load transfer could be less. The same maybe true for thinner sections. There may be wider cracks and less aggregate interlock effect in thinner slabs and consequently there would be less load transfer compared to thicker slabs. It is also expected that undoweled sections would have lower load transfer. It should be noted that SDDOT considers the load transfer to be a concern when it is less than 80%.

An attempt was made to measure the actual crack widths across various joints and other cracks in the unjointed sections by fixing P.K. Nails and measuring the distance between them at various times. (The results are given in Appendix C and were discussed earlier). The devices used for measuring the crack width had an accuracy (least count) of only 1.6mm (0.0625 inch) and this accuracy was not adequate to measure the crack width accurately enough for the analysis. Hence a definite conclusion could not be made regarding the exact crack width at joints with different joint spacings. A more accurate crack measuring device with a least count of 0.003-mm (0.0001-inch) would have given us the needed information.

An interesting comparison is the elastic modulus values for concrete calculated back calculated by the FWD. The elastic modulus values for concrete at the control section (A) and NMFRC sections (B to H) are given in the table on page F4. The calculated modulus values for the control section A was 29628.6 Mpa (4294 Ksi) whereas the concrete modulus values for the NMFRC sections (B to H) were consistently lower with an average value of 25205.7 Mpa (3653 ksi). There is a 15 percent less modulus for the NMFRC with same compressive strength. There seems to be the same difference (15 percent) in the load transfer values for equivalent NMFRC sections.

It has been well documented in this project, other projects, and various other researchers all over the world that there is no difference in the elastic modulus of the plain (control) concrete and fiber reinforced concrete (1to 31). Therefore some changes must be made in the equations used and interpretation of results obtained from the falling weight deflectometer test in regard to FRC. When the same concrete modulus values are obtained in the calculations for both plain and FRC for same compressive strength, then only the test results obtained from the falling weight deflectometer test are reliable and acceptable.

Riding Quality

Tests for International Roughness Index (IRI) and South Dakota Index (SDI) were conducted by SDDOT engineers and the results are included in Appendix F. For new paving, the IRI should be 1.2 or less and the SDI should be 4.4 or greater. The results indicate that these criteria were met by both NMFRC and PJCP pavements. There was no significant difference in the riding quality between the PJCP (control) and NMFRC pavements. There was also no difference shown between the jointed and unjointed NMFRC pavements.

The PI had driven on this highway after the pavement was constructed until September '98 and he could not feel any difference in the riding quality between the doweled PJCP and NMFRC pavements. He had also questioned a number of DOT employees and others who used this highway. They did not find any difference either.

Faulting Data

The Faulting data supplied are included in the Appendix F. The test results indicate that there is no significant difference between the doweled PJCP and the NMFRC pavements. There is also no clear difference between the jointed and unjointed NMFRC pavements.

Research Task 7: Recommend design, testing, and construction guidelines for using NMFRC in full depth pavements based on results from the test sections.

Based on the observed performance of the NMFRC full depth pavements constructed, the following guidelines are recommended.

Since no other design procedure is available currently, it is recommended that the NMFRC pavement thickness can be designed by modifying the PCA pavement thickness design method using appropriate NMFRC test data.

It is recommended that the same control tests conducted in this project for NMFRC fresh concrete namely, the slump, unit weight, air content, and test for actual fiber content could be specified. The standard test procedures are described in Task 3. It is also recommended, that the concrete temperature, the ambient temperature, humidity, and the wind velocity should be recorded.

Field samples should be collected and cured using ASTM standard procedures, and the following hardened concrete performance tests should be conducted for NMFRC at 28–days: compressive strength, elastic modulus, flexural strength (modulus of rupture), fatigue strength, and toughness values(ASTM and Japanese standards). The standard test procedures to be followed are described in Task 3.

The same construction procedures currently used for plain concrete for mixing, transporting, placing, consolidating, finishing, tining and curing could be used for

NMFRC. Some additional mixing time be required for NMFRC which should be determined by field trials.

The procedure for adding fibers to the mix may vary depending on the field conditions. This could be determined by trial mixing.

Research Task 8: Using cost data available from SDDOT and others, compare the performance and life-cycle costs of NMFRC pavement and plain concrete pavements. NMFRC cost estimates should assume that its use becomes common construction practice and is no longer experimental

The life cycle cost analysis was done using the computer program available with the SDDOT. This analysis was done with the help of Dan Strand (project monitor) and Gill Hedman (Materials and Surfacing). Two life cycle cost estimating worksheets are included in appendix F.

One sheet uses an analysis period of 40 years, whereas, the other uses 60 years. For each of the analysis periods, "Alternative 1" indicates costs which are associated with SD's doweled PJCP and "Alternative 2" indicates costs for the NMFRC.

The 40 year analysis

The "initial costs" for both alternatives were computed based on a per mile cost for concrete 28 feet wide (28ft x 5280ft/mile = 147,840 ft^2/mile = 16427 yd^2/mile). The cost for the doweled PJCP was \$15.35/yd^2 and the cost for the 8" NMFRC was \$ 28.90/yd^2. These prices came from the actual unit bid items for the US83 paving project.

The "periodic costs" for Alternative 1 are applied at a life and an amount that may seem reasonable based on the past experience.

The "annual costs" for alternative 1 are based on past experience.

With no periodic or annual costs for the NMFRC, the "% Difference From Lowest LCC Alternative" is high (61%) for the 40 year analysis. This clearly indicates that the high initial cost does not make NMFRC a viable alternative.

The 60 Year Analysis

Because of the improved structural properties of the NMFRC, we would hope that the life of the concrete would be increased. Also, because of the large "% Difference From Lowest LCC Alternative" in the 40-year analysis, the life of the NMFRC would have to increase by a substantial margin so that a break-even point between the two alternatives can be reached. Therefore, due to a 50% increase in the fatigue life of the NMFRC it was assumed that the life of the concrete might be extended to 60 years.

In order to extend the life of the PJCP to 60 years, two additional "periodic costs" were added. 1) A 3-inch "AC Overlay" would be placed at year 40 with a cost of \$200,000/mile. 2) A "Mill & AC Overlay" would be completed at year 50 with a cost of \$150,000/mile (1" milled and 2" put back).

Again, in order to extend the life of " Alternative 1" to 60 years, an additional annual cost of \$2000/mile was added between years 41 and 60.

Having added 20 years to the doweled PJCP along with the necessary " periodic costs" and "annual costs" the "% Difference From Lowest LCC Alternative" remains too high (31%) for the NMFRC to even be considered. No "periodic costs" or "annual costs" were incurred for the NMFRC. (This assumption is not possible, but was assumed as an absolute best case for NMFRC).

The above analysis did not take into account the inconvenience caused to the public by closing the highway and /or slowing of the traffic during the numerous repair and maintenance operations in the doweled PJCP.

Research Task 9: Provide an interim report 90 days after the test sections are constructed. The interim report should document the construction evaluation, material properties, early performance of the test and control sections, and should include sketches showing relative locations of each test section along with descriptive text. The interim report should be submitted 90 days after the test sections construction completion date.

An interim report was submitted to the SDDOT, 90 days after the test sections were constructed. The report included all the requested information.

Research Task 10: Submit a final report summarizing relevant literature, research methodology, test results, specifications, design standards, conclusions, and recommemdations.

The final report has been submitted which summarizes all aspects of this study.

Research Task 11: Make an executive presentation to the SDDOT Research Review Board summarizing the findings and conclusions.

An executive presentation summarizing the finding and conclusions of this project was given at the November 22, 1998 Research Review Board Meeting.

Conclusions

1. The ability of using NMFRC in the construction of full depth pavements has been established. The same construction techniques and construction equipment without any permanent modification could be used in construction of full depth pavements using NMFRC. However a large diameter plastic tube was added to facilitate the introduction of the fibers in the batching process and the number of people had to be increased to add the fibers.
2. The addition of polyolefin fibers at 14.8 kg/m^3 (25 lbs./cu.yd.) enhanced the structural properties of concrete. There was a considerable increase in toughness, impact, fatigue, endurance limit, and post crack load carrying capacity. The most important contribution due to the addition of fibers to concrete is the change in the mode of failure from a dangerous brittle failure to a more desirable ductile failure when subjected to compression, flexure, impact and fatigue loads.
3. It is possible to have longer joint spacings in NMFRC pavements. In the short duration of 3 years, the inspections had shown that there was no distress such as excessive cracking, spalling and fatigue cracking at the joints in pavement segments with 7.6m (25-ft.) joint spacings and 10.6m (35-ft.) joint spacings.

   The unjointed pavement cracked at approximately 26m (85.2-ft.) spacings. Once these cracks formed, no additional cracks formed and the crack widths seemed to have remained the same during the inspection period.
4. The inspections had shown that there was no difference in the behavior of the joints for the thicker (203 mm or 8 in.) pavement and thinner (165mm or 6.5 in.) pavements. The random cracks in the unjointed section behaved in a similar manner to that of the joints in thin and thick NMFRC sections.
5. The post construction performance of the control and NMFRC slabs was satisfactory. Once the cracks formed in the unjointed NMFRC pavement section, the polyolefin fibers helped to contain the crack propagation and to resist the widening of cracks.
6. No difference in the riding quality could be established between PCC and NMFRC pavements with different joint spacings, different thicknesses, and doweled and undoweled sections. The measured International Roughness Index (IRI) and South Dakota Index (SDI) for both control and NMFRC pavements were satisfactory for the

new pavement criteria. There was no significant difference in these indices for the control section and various sections of the NMFRC pavement.

7. The Falling Weight Deflectometer Test has shown that the load transfer was less in all the NMFRC sections compared to the control section. In general the load transfer was less in sections with longer joint spacings, smaller thickness, and undoweled. However the test had also shown that the elastic modulus value for NMFRC was 15 percent less than that of control concrete with the same strength. This result is contradictory to all the known facts and therefore the results are not totally acceptable.

Based on the FWD data (if it were assumed to be correct), dowels should be used at control joints to increase the load transfer across the joints. This was shown by the low percentage of load transfer for all the random cracks and the control joints which did not have dowels. It should be noted that the number of fibers crossing the random cracks and control joints were reduced due to saw cutting. Therefore, the ability of the fibers to transfer load across random cracks and control joints was reduced. 8. The life cycle cost analysis has shown that NMFRC with its high initial cost is not a favorable material for the construction of full depth pavements. However when the cost of fiber becomes cheaper, then this may be a viable material for the construction of full-depth pavement.

Recommendations

1. When initial cost is not a deciding factor, and when longer joint spacings, thinner sections and more efficient performance are the requirements, then NMFRC full depth pavements could be used in special cases.
2. The equations and empirical constants used in the evaluation of load transfer in the Falling Weight Deflectometer Test should be analyzed and modified for use with NMFRC pavement so that they reflect correctly the known facts about the concrete modulus values.
3. A more accurate method should be used for measuring crack-widths and joint openings, and their changes with seasons and with time.

4. Since no other design procedure is available currently, it is recommended that the NMFRC pavement thickness can be designed by modifying the PCA pavement thickness design method using appropriate NMFRC test data.
5. It is recommended that the following control tests be conducted for NMFRC fresh concrete: slump, unit weight, air content, and fiber content.
6. It is recommended, that the concrete temperature, the ambient temperature, humidity, and the wind velocity be recorded during the placing of the concrete.
7. It is recommended that field samples be collected and cured using ASTM standard procedures and that the following hardened concrete performance tests be conducted for NMFRC at 28–days: compressive strength, elastic modulus, flexural strength (modulus of rupture), fatigue strength, and toughness values(ASTM and Japanese standards).
8. It is recommended that the same construction procedures for mixing, transporting, placing, consolidating, finishing, tining, and curing used for full depth paving with plain concrete be used for NMFRC. NOTE: Some additional mixing time may be required for NMFRC which should be determined by field trials.
9. Based on the observed short term performance of the joints in the NMFRC, it is recommended that longer joint spacings could be used for NMFRC pavement. For thicker full depth NMFRC pavements (203mm (8.0in)) a 15.25m(50 ft) joint spacing is suggested, and for thinner full depth NMFRC pavements (165mm (6.5in)) a 10.7m (35ft) joint spacing is recommended. This recommendation is based on the assumption that the FWD data did not truly reflect the load transfer in NMFRC pavements.
10. It is recommended that SDDOT's standard saw depths used for transverse and longitudinal joints be used for NMFRC full depth paving.
11. Based on the FWD results, it is recommended that control joints for NMFRC full depth pavement use dowel baskets as a mehtod to transfer load across joints.

REFERENCES

1. ACI Committee 544, "State-of-the-Art Report on Fiber Reinforced Concrete", Report ACI 544 IR-82, Concrete International Design and Construction, May 1982.
2. Ramakrishnan, V., "Recent Advancements in Concrete Fiber Composites", ACI-Singapore Chapter – Special Publication, July 19, 1993.
3. Ramakrishnan, V., "Concrete Fiber Composites for the Twenty-First Century," Real World Concrete, Proceedings of R. N. Swamy Symposium, Fifth CANMET/ACI International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Milwaukee, June 4-9, 1995, pp. 111-143.
4. Ramakrishnan, V., "High Performance Fiber Reinforced Concretes," Application of High Performance Concrete Including Marine Structures, National Science Foundation (NSF) and Australian Research Council (ARC) – Sponsored USA-Australia Workshop, Sydney, Australia, August 20-23, 1997, pp. 2-31.
5. Ramakrishnan, V., "Evaluation of Non-Metallic Fiber Reinforced Concrete in PCC Pavements and Structures," *Report No. SD94-04-I, South Dakota Department of Transportation*, Pierre, SD, 1995, 319 pages.
6. Sivakumar Arunachalam, "Performance Characteristics of Polyolefin Fiber Reinforced Concrete," M. S. Thesis, South Dakota School of Mines and Technology, Rapid City, South Dakota 57701, 1994.
7. Ashokkumar Maragondanahalli, "Influence of Various Parameters on the Performance and Properties of 3M Fiber Reinforced Concretes," M. S. Thesis, South Dakota School of Mines and Technology, Rapid City, South Dakota 57701, 1994.
8. Bjarte Nesse, "High Volume Non-Metallic Fiber Reinforced Concrete", M. S. Thesis, South Dakota School of Mines & Technology, Rapid City, South Dakota 57701, 1996.
9. Roar Nakling Martinsen, "Performance Characteristics of Polyolefin Fiber Reinforced Concrete (A Comparison and Evaluation of Testing Methods)," M. S. Thesis, South Dakota School of Mines and Technology, Rapid City, South Dakota 57701, 1995.

10. Morgan, D.R.; and Rich, L.D., "Polyolefin Fiber Reinforced Wet-Mix Shotcrete," ACI/SCA International Conference on Sprayed Concrete and Shotcrete, World Innovations for the 21st Century, Edinburgh, Scotland, Sept. 10-11, 1996.
11. Morgan, D.R.; Lobo, A.; and Rich, L., "Repair of Berth Faces at the Port of Montreal with Fiber Reinforced Shotcrete," Seminar on High Performance Fiber Reinforced Concrete in Infrastructural Repair and Retrofit, ACI, New Orleans, Nov. 3-4, 1996.
12. Ramakrishnan, V., "Performance Characteristics of Polyolefin Fiber Reinforced Concrete," *Materials for the New Millennium*, Proceedings of the Fourth Materials Engineering Conference, Washington, D.C., November 10-14, 1996, pp. 93-102
13. Balaguru, P.; Kurtz, S.; and Rudolph, J., "Shrinkage Cracking Characteristics of Polyolefin Fiber Reinforced Concrete," Report Submitted to 3M Company, St. Paul, Minnesota, December, 1996.
14. Banthia, N.; Yan, C.; and Lee, W.Y., "Restrained Shrinkage Cracking in Fiber Reinforced Concrete with Polyolefin Fibers," Department of Civil Engineering, The University of British Columbia, Vancouver, BC, V6T 1Z4, Canada.
15. Ramakrishnan, V., and MacDonald, C.N., "Durability Evaluations and Performance Histories of Projects Using Polyolefin Fiber Reinforced Concrete," *Durability of Concrete*, Proceedings of the Fourth CANMET/ACI International Conference, Sydney, Australia, 1997, pp. 665-680.
16. Ramakrishnan, V., "A New Material (Polyolefin Fiber Reinforced Concrete) for the Construction of Pavements and White-Topping of Asphalt Roads," Proceedings of the Sixth International Purdue Conference on Concrete Pavement: Design and Materials for High Performance, Indianapolis, Nov. 18-21, 1997, pp. 119-130.
17. Ramakrishnan, V., "Performance Characteristics and Applications of High – Performance Synthetic Fiber Reinforced Concretes," Proceedings of the International Workshop on High Strength Concrete and Structural Strengthening, Singapore, Nov. 29, 1997, pp. 33-54.
18. Ramakrishnan, V., "Structural Applications of Polyolefin Fiber Reinforced concrete," American Concrete Institute, Spring Convention, Session on "Structural Application

of Fiber Reinforced Concrete," Seattle, Washington, April 6-11, 1997. (Accepted for publication in the proceedings.)

19. Ramakrishnan, V., "Application of a New High Performance Polyolefin Fiber Reinforced Concrete in Transportation Structures," TCDC Workshop on Advances in High Performance Concrete Technology and its Applications, Government of India, Structural Engineering Research Center and United Nations UNDP, April 16-18, 1997, Madras, India.
20. Ramakrishnan, V., "Application of a New High Performance Polyolefin Fiber Reinforced Concrete in Transportation Structures" Proceedings of the PCI/FHWA International Symposium on High Performance Concrete, New Orleans, Louisiana, October 20-22, 1997.
21. Ramakrishnan, V., Strand, D., MacDonald, C.N., "Performance Characteristics of a New Material (Polyolefin Fiber Reinforced Concrete) for Repair and Rehabilitation of Bridges and Pavements," Proceedings: A Research–To–Practice Symposium on Research and Rehabilitation of Bridges and Pavements, Warwick, Rhode Island, May 1-3, 1996, pp. 61-75.
22. Strand, D.; MacDonald, C.N.; Ramakrishnan, V.; Rajpathak, V.N., "Construction Applications of Polyolefin Fiber Reinforced Concrete," *Materials for the New Millennium*, Proceedings of the Fourth Materials Engineering Conference, Washington, D.C., November 10-14, 1996, pp. 103-112.
23. Sprinkel, M.; Ozyildirim, C.; Hladky, S.; and Moen, C., "Pavement Overlays in Virginia," Proceedings of the Sixth International Purdue Conference on Concrete Pavement Design and Materials for High Performance, Vol. 2, Purdue University, Nov. 18-21, 1997, pp. 217-230.
24. Ramakrishnan, V., "Demonstration of Polyolefin Fiber reinforced Concrete in Bridge Replacement" Final Report 5095-22, South Dakota Department of Transportation, Pierre, S.D, December 1997.
25. Spadea, G., and Bencardino, F., "Behavior of Fiber-Reinforced Concrete Beams under Cyclic Loading," *Journal of Structural Engineering*, Vol. 123, No. 5, May 1997, pp. 660-668.

26. Trottier, J.F., Morgan, D.R., and Forgeron, D., " Fiber Reinforced Concrete for Exterior Slabs-on-grade, part 1", Concrete International, Vol. 19, No. 6, June 1997, pp. 35-39.
27. Bakht, B., Mufti, A., "F. R. C. Deck Slabs Without Tensile Reinforcement," *Concrete International,* Feb. 1996, pp. 50-55.
28. Billy D. Neeley., and Edward F. O'Neil., "Polyolefin Fiber Reinforced Concrete," *Materials for the New Millennium*, Proceedings of the Fourth Materials Engineering Conference, Washington, D.C., November 10-14, 1996, pp. 113-122.
29. Li Fang., and Christian Meyer., "Biaxial Low-Cycle Fatigue Behavior of Steel Fiber Reinforced Concrete," *Materials for the New Millennium*, Proceedings of the Fourth Materials Engineering Conference, Washington, D.C., November 10-14, 1996, pp. 436-445.
30. Ramakrishnan,V., C.Meyer,A.E.Naaman, "Cyclic Behavior ,Fatigue Strength, Endurance Limit and Models for Fatigue Behavior of FRC", Chapter 4,High Performance Fiber Reinforced Cement Composites 2,E& FN Spoon ,New York,1996.
31. Banthia, N.; Yan, C.; and Lee, W.Y., "Restrained Shrinkage Cracking in Fiber Reinforced Concrete with Polyolefin Fibers," Proceedings of the Fifth International Conference on Structural Failure, Durability and Retrofitting, Singapore, Nov. 27-28, 1997, pp. 456-470.

APPENDIX A

Details of Laboratory Trial Batches

DOT Trial Batches for Full depth pavement

Table A1: Mixture Proportions

Mixture #	Mixture Proportions lb/cu. yd.						
	Cement	Fly Ash	Coarse Agg.	Fine Agg.	Water	Fiber	AEA oz/cu. yd
DOT-T3	510	112	1417	1417	264	25	8
DOT-T4	510	112	1417	1417	264	25	8
DOT-T6	510	112	1417	1417	264	25	8

Table A2: Fresh Concrete Properties

Mixture #	Room Temp. Humidity		Conc. Temp.	Unit Weight	Air Content	Slump	Vebe Slump	Time
	(°F)	(%)	(°F)	(lb/cu ft)	(%)	inches	inches	Sec.
DOT-T3	70	45	74.9	140.4	6.0	3.5	1.875	2
DOT-T4	80	40	80.2	141.7	5.6	2.25	1.875	2
DOT-T6	80	30	79.1	142.4	4.4	1.25	0.875	7

Table A3: Number of Specimens

Mixture #	Number of Specimens		
	Beams	**Cylinders**	**Impact**
DOT-T3	8	6	15
DOT-T4	-	14	-
DOT-T6	23	2	-

Conversion table:
1 inch = 25.4 mm
1 pcf = 16.02 kg/cu m
1 sq. in. = 645.2 sq mm

1 psi = 0.006895 Mpa
1 pcy = 0.5933 kg/cu m
1 lb = 0.4536 kgf = 4.448 N

°F to °C: T(°C) = [T(°F) - 32]/1.8
1 inch-pound = 0.1130 Nm

Table A4: Compressive Strength

Specimen #	Age (days)	Date of Testing	Length (inches)	Diameter (inches)	Area (Sq. Inches)	Unit Weight (pcf)	Compressive Strength (psi)
DOT-T3-C1	7	07/01/96	12.135	5.990	28.184	145.511	4770
DOT-T3-C2	7	07/01/96	12.091	6.002	28.297	145.457	4770
DOT-T3-C3	7	07/01/96	12.185	5.995	28.231	144.673	4710
Average							**4750**
DOT-T3-C4	28	07/22/96	12.073	5.980	28.086	144.219	5875
DOT-T3-C5	28	07/22/96	12.081	5.998	28.255	143.769	5930
DOT-T3-C6	28	07/22/96	12.118	5.974	28.030	144.480	5710
Average							**5840**

Table A5: First Crack Strength and Maximum Flexural Strength

Specimen #	Age (Days)	First Crack			Maximum Load (lbs)	Flexural Strength (psi)
		Load (lbs)	Deflection (inches)	Stress (psi)		
DOT-T3-B1	7	2789	0.0005	471	3134	529
DOT-T3-B2	7	2725	0.0003	489	2937	527
DOT-T3-B3	7	2819	0.0006	480	2938	501
DOT-T3-B4	7	2788	0.0009	487	3184	556
Average				**480**		**528**
DOT-T3-B5	28	2526	0.0006	437	2809	486
DOT-T3-B6	28	3025	0.0014	553	3235	592
DOT-T3-B7	28	3422	0.0005	558	3422	558
DOT-T3-B8	28	2730	0.0010	494	2785	504
Average				**511**		**535**

Conversion table:
1 inch = 25.4 mm
1 pcf = 16.02 kg/cu m
1 sq. in. = 645.2 sq mm
1 psi = 0.006895 Mpa
1 pcy = 0.5933 kg/cu m
1 lb = 0.4536 kgf = 4.448 N
°F to °C: T(°C) = [T(°F) - 32]/1.8
1 inch-pound = 0.1130 Nm

Table A6: Japanese Standard - Toughness & Equivalent Flexural Strength

Specimen #	Age (Days)	Toughness (Inch-lbs)	Equivalent Flexural Strength (psi)
DOT-T3-B1	7	115	242
DOT-T3-B2	7	150	337
DOT-T3-B3	7	124	264
DOT-T3-B4	7	144	314
Average		**133**	**289**
DOT-T3-B5	28	150	325
DOT-T3-B6	28	126	288
DOT-T3-B7	28	142	290
DOT-T3-B8	28	149	336
Average		**142**	**310**

Table A7: ASTM Toughness Indices

Specimen #	Age (Days)	First Crack Toughness (inch-lbs)	Toughness Indices			Toughness Ratios	
			I5	I10	I20	I10/I5	I20/I10
DOT-T3-B1	7	0.76	5.01	9.77	18.21	1.95	1.86
DOT-T3-B2	7	0.64	3.74	7.13	13.54	1.91	1.90
DOT-T3-B3	7	1.39	3.50	6.47	11.84	1.85	1.83
DOT-T3-B4	7	1.46	4.77	9.26	16.85	1.94	1.82
Average		**1.06**	**4.26**	**8.16**	**15.11**	**1.91**	**1.85**
DOT-T3-B5	28	1.05	4.15	7.94	14.87	1.91	1.87
DOT-T3-B6	28	3.05 *	3.86	7.09	12.29	1.83	1.73
DOT-T3-B7	28	1.51	3.22	5.87 *	10.70 *	1.82	1.82
DOT-T3-B8	28	1.63	4.36	8.28	15.11	1.90	1.83
Average		**1.40**	**3.90**	**7.77**	**14.09**	**1.87**	**1.81**

* Omitted during calculations

Conversion table:

1 inch = 25.4 mm
1 pcf = 16.02 kg/cu m
1 sq. in. = 645.2 sq mm

1 psi = 0.006895 Mpa
1 pcy = 0.5933 kg/cu m
1 lb = 0.4536 kgf = 4.448 N

°F to °C: T(°C) = [T(°F) - 32]/1.8
1 inch-pound = 0.1130 Nm

Table A8: 28 Days Impact Strength

Specimen #	Number of Blows	
	First Crack	Failure
DOT-T3-I1	96	400
DOT-T3-I2	93	501
DOT-T3-I3	108	373
DOT-T3-I4	107	385
DOT-T3-I5	333	600
DOT-T3-I6	200	330
DOT-T3-I7	70	213
DOT-T3-I8	276	538
DOT-T3-I9	83	343
DOT-T3-I10	62	347
DOT-T3-I11	108	511
DOT-T3-I12	124	441
DOT-T3-I13	323	612
DOT-T3-I14	46	452
DOT-T3-I15	88	331

Fig. A1: Comparison of Compressive Strength for Different Specimens of the Same Batch

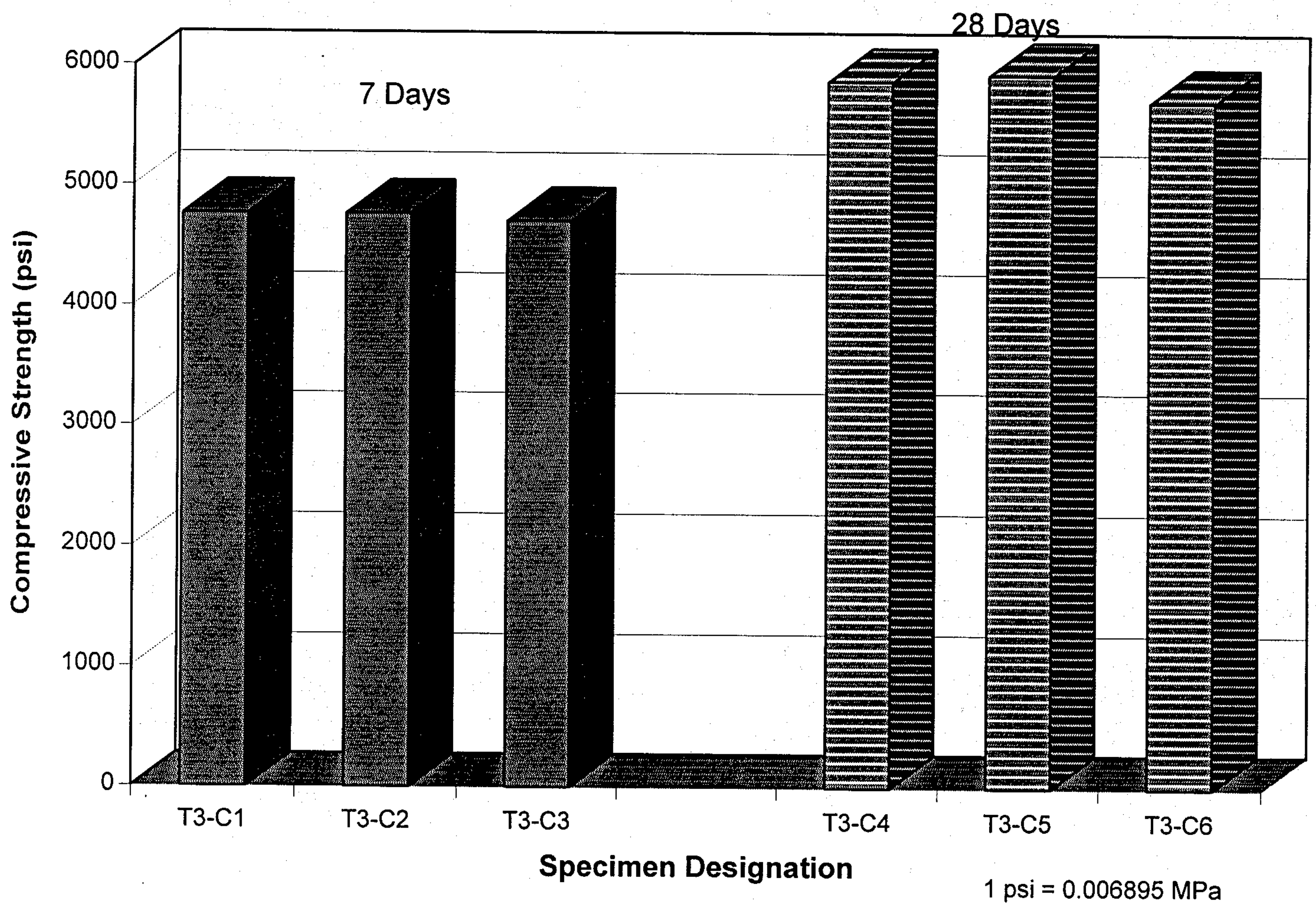
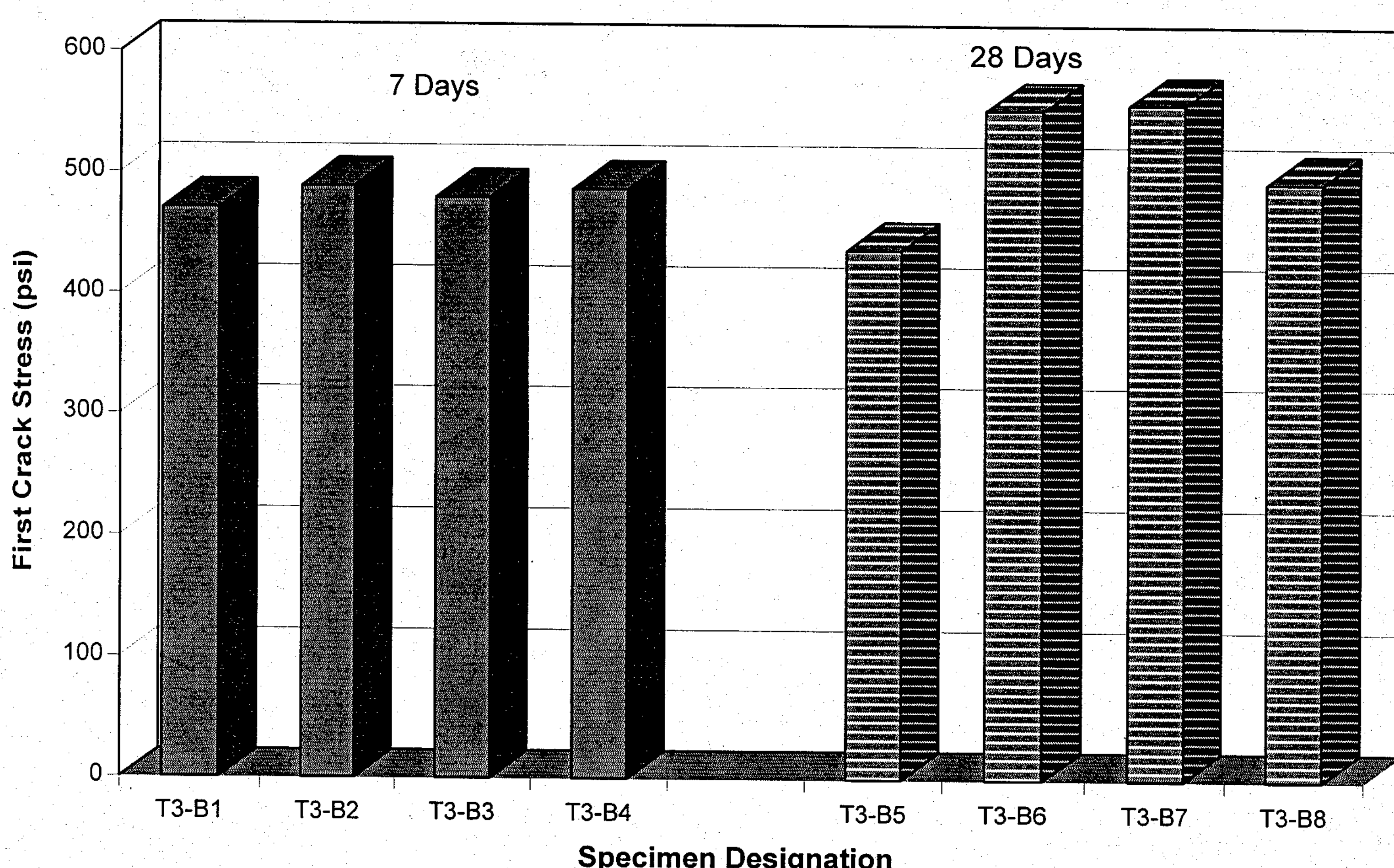
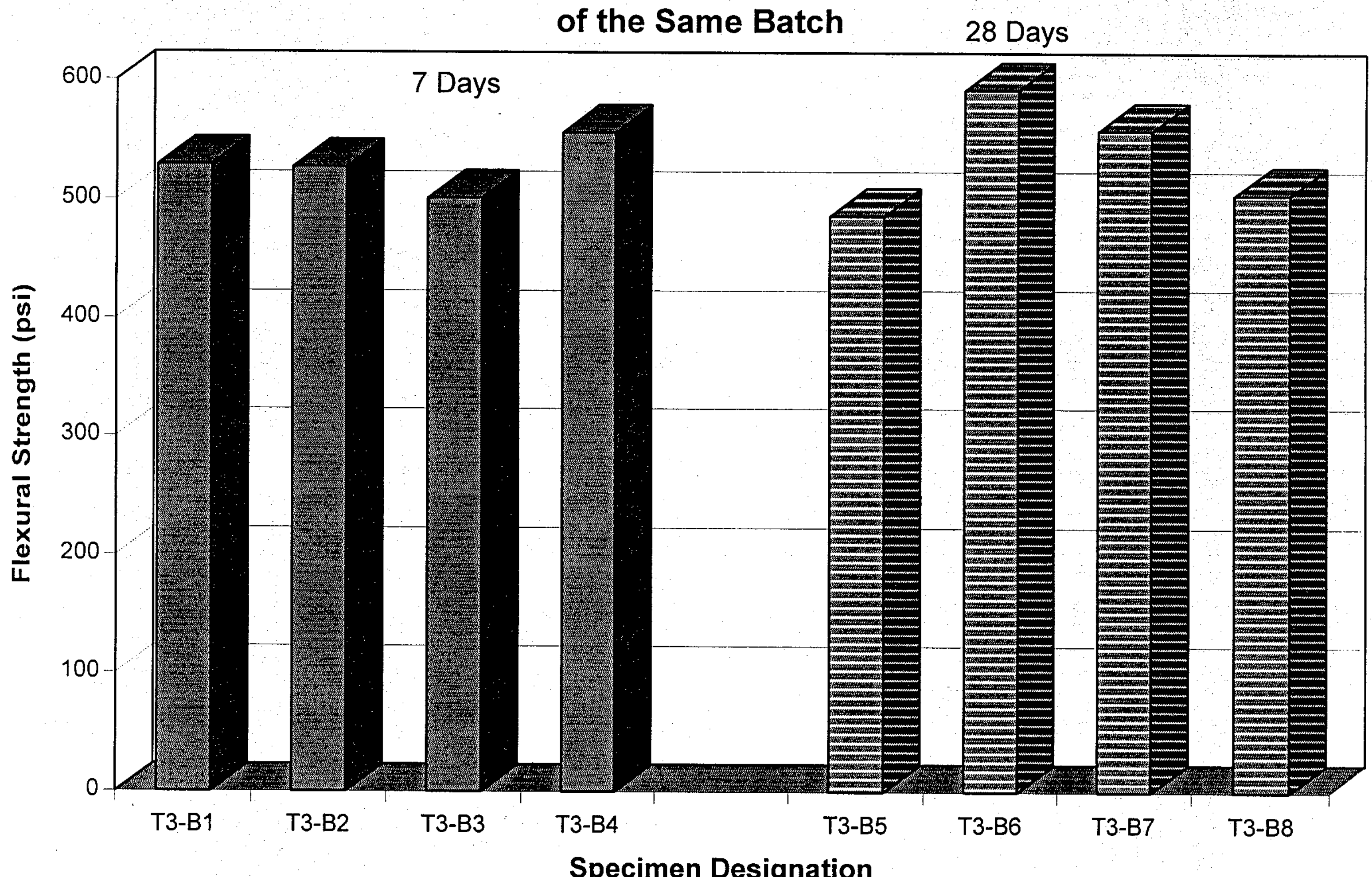


Fig A2: Comparison of First Crack Stress for Different Specimens of the Same Batch



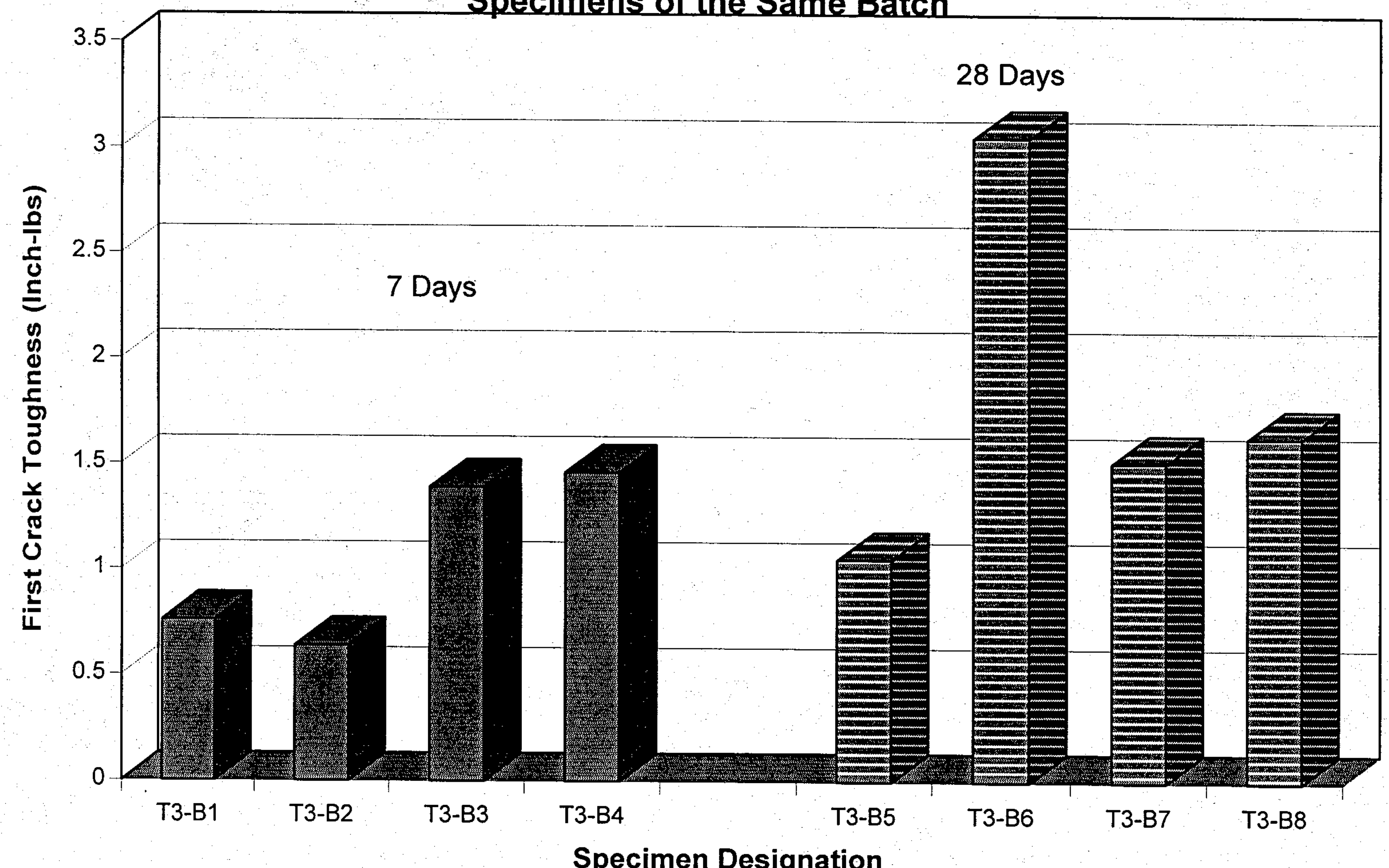
1 psi = 0.006895 MPa

Fig A3: Comparison of Flexural Strength for Different Specimens of the Same Batch



1 psi = 0.006895 MPa

Fig A4: Comparison of ASTM First Crack Toughness for Different Specimens of the Same Batch



T3-B6 was Omitted during Calculations

1 Inch-lb = 0.1130 Nm

Fig A5: Comparison of ASTM Toughness Indices for Different Specimens of the Same Batch

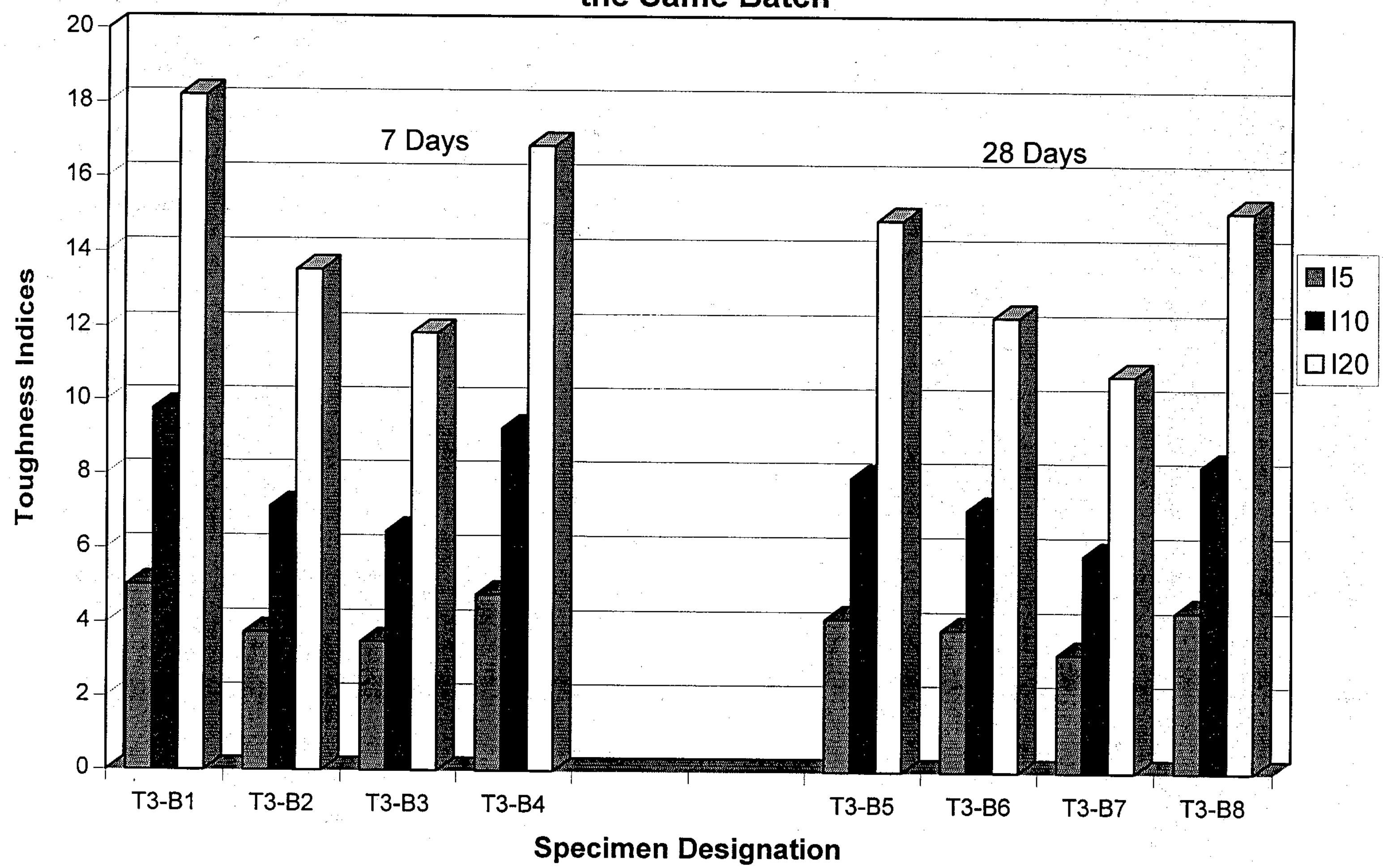


Fig A6: Comparison of ASTM Toughness Ratios for Different Specimens of the Same Batch

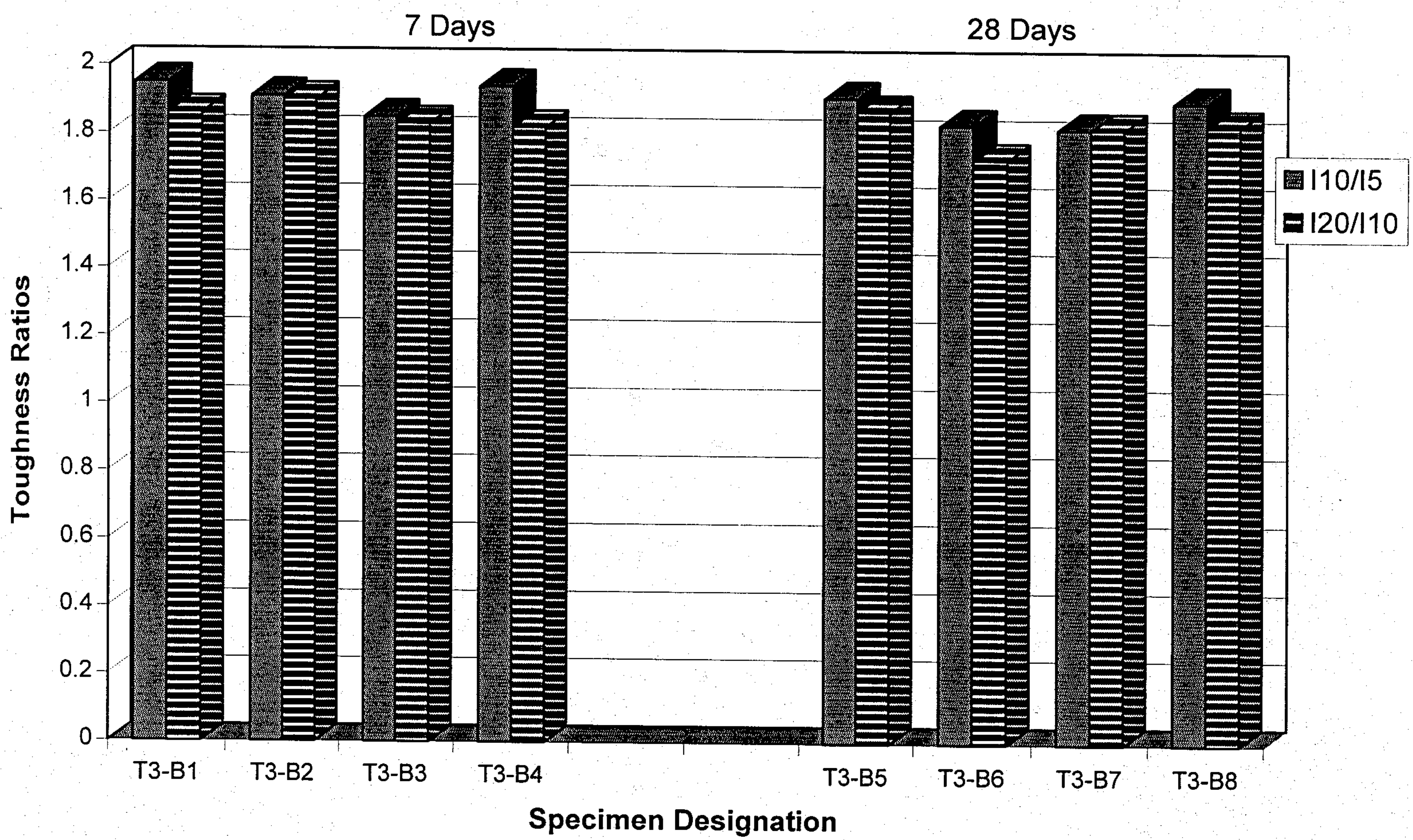


Fig A7: Comparison of Japanese Toughness for Different Specimens of the Same Batch

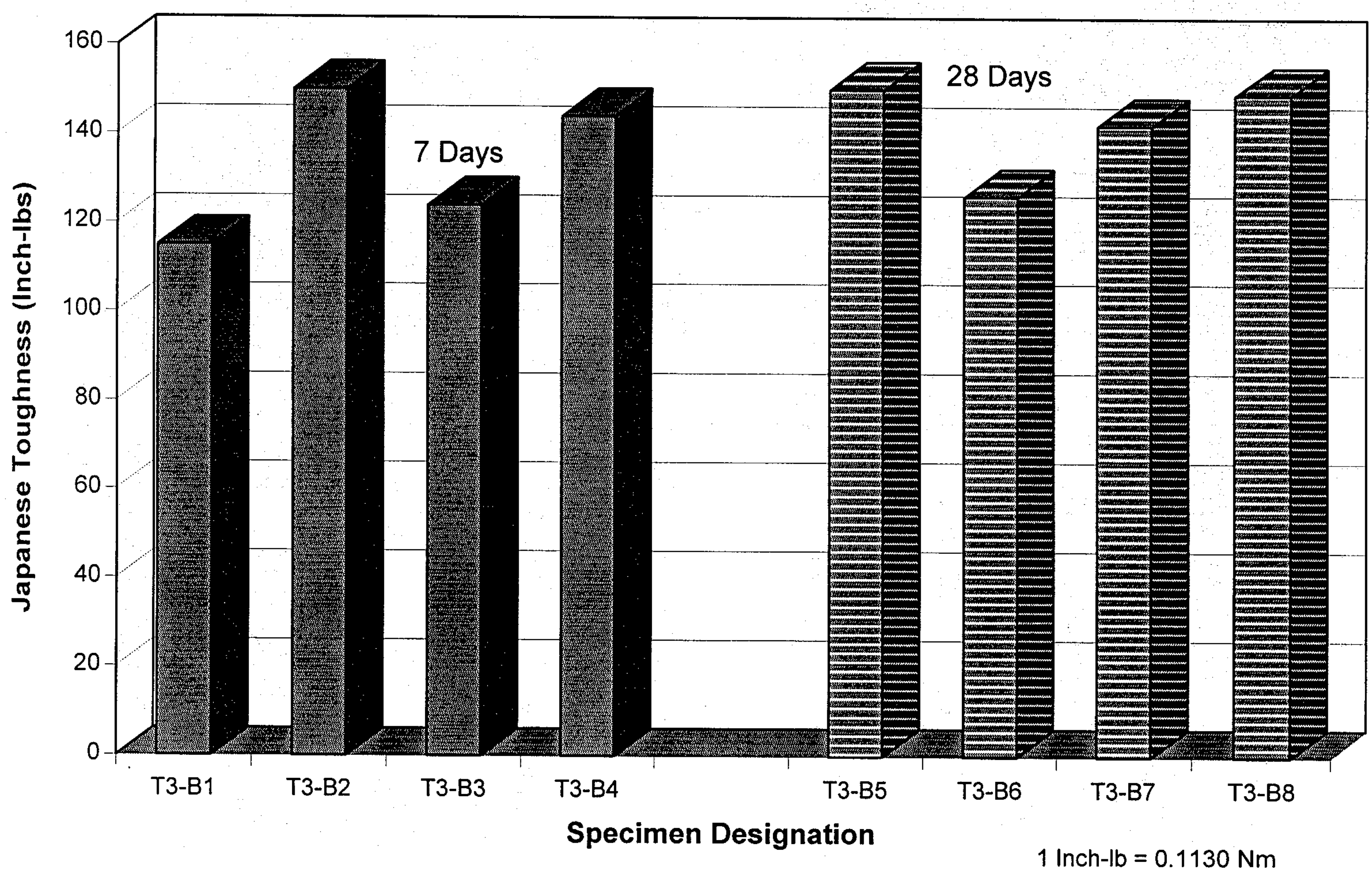
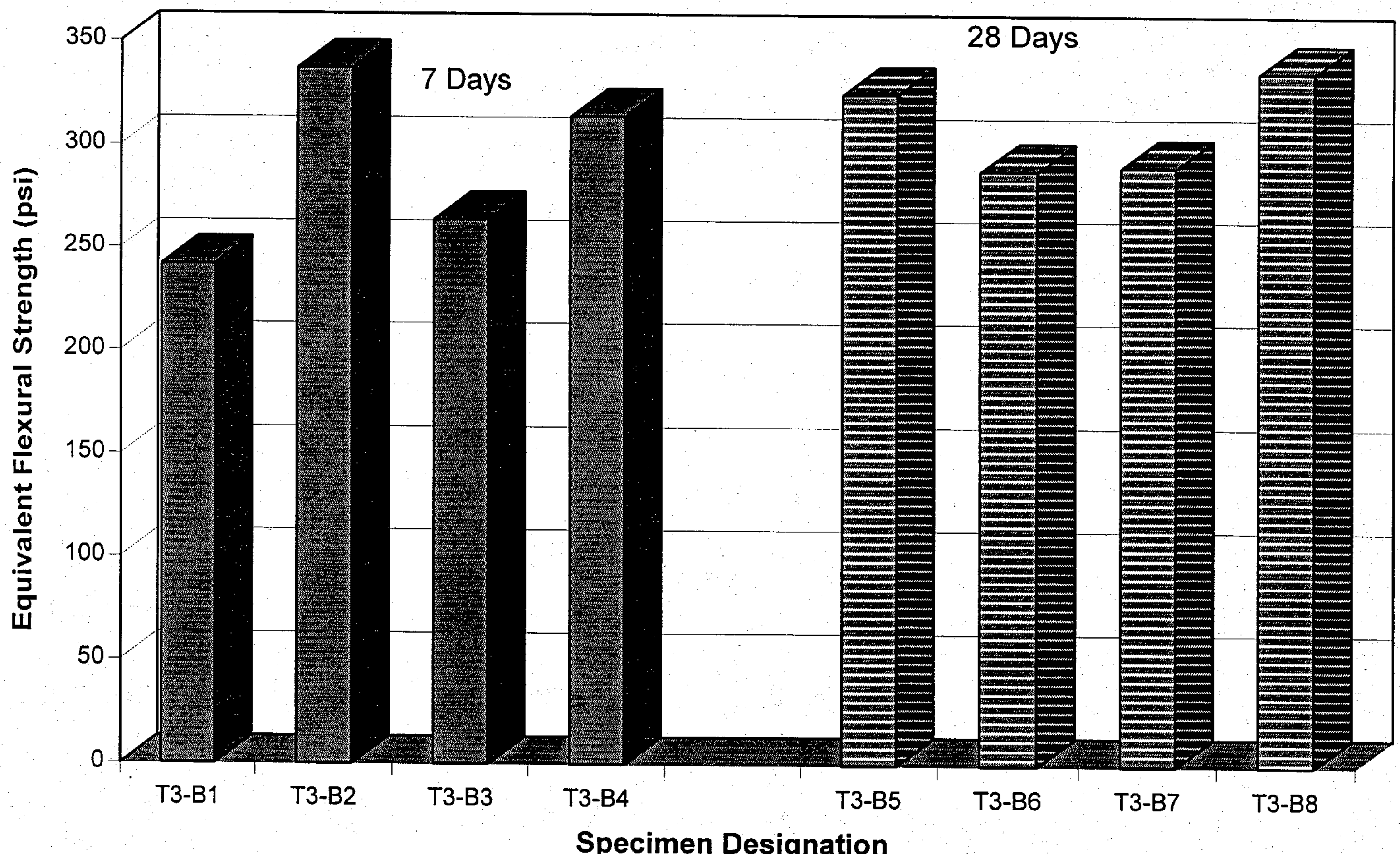


Fig A8: Comparison of Japanese Standard Flexural Strength for Different Specimens of the Same Batch



1 psi = 0.006895 MPa

APPENDIX B

Details of Fresh and Hardened Concrete Properties for NMFRC Full-Depth Pavement

Specimens made from the paving done on August 15, 1996 for Full-Depth Pavement on Highway 83 (North Bound Lane)

Table B1: Fresh Concrete Properties

Mixture #	Time	Ambient Temp.	Ambient Humidity	Conc. Temp. *50°-90°	Unit Weight	Air Content *6.5+/- 1.5%	Slump *2"Max	Actual Fiber Content *25lbs/yd^3
		(°F)	(%)	(°F)	(lb/cu ft)	(%)	Inches	(lb/cu yd)
P1	7:45	55	80	78.9	145.848	6.0	1/2	-
(8/15/96)	8:30	60	80	76.6	145.024	6.4	1-3/4	-
	9:10	63	75	79.4	142.964	7.5	2	25.45
	9:45	72	65	78.9	143.788	8.3	1-1/2	-
	10:40	84	50	81.5	145.848	6.9	3/4	-
	12:05	86	35	81.6	144.612	7.8	1-1/4	23.68
	13:30	87	33	84.7	145.024	7.0	1-1/4	23.44
	15:00	92	31	88.4	144.200	7.3	1-1/4	-
	16:00	92	30	86.5	145.024	7.8	3/4	-
	17:00	90	30	85.7	146.260	6.2	3/4	-
	17:50	84	38	85.2	144.200	7.6	1-3/4	-
	18:15	84	38	85.7	150.380	7.0	2-1/4	-

* DOT Specification Range

Table B2: Number of Specimens

Mixture #	Number of Specimens		
	Beams	**Cylinders**	**Impact**
P1	10	6	20
P1-Fatigue	25	3	-
P1-Control	4	3	-

Conversion table:
1 inch = 25.4 mm
1 psi = 0.006895 Mpa
°F to °C: T(°C) = [T(°F) - 32]/1.8
1 pcf = 16.02 kg/cu m
1 pcy = 0.5933 kg/cu m
1 inch-pound = 0.1130 Nm
1 sq. in. = 645.2 sq mm
1 lb = 0.4536 kgf = 4.448 N

Table B3: Compressive Strength

Specimen #	Age (days)	Date of Testing	Length (inches)	Diameter (inches)	Area (Sq. In.)	Unit Weight (pcf)	Static Modulus (psi)	Comp. Str. (psi)
P1-C1	7	08/22/96	12.186	6.000	28.274	144.942	4.24×10^6	3960
P1-C2	7	08/22/96	12.162	6.010	28.369	144.491	4.23×10^6	3880
P1-C3	7	08/22/96	12.141	6.013	28.397	143.345	4.23×10^6	3750
Average								**3865**
P1-C4	28	09/12/96	12.110	5.985	28.133	143.285	4.27×10^6	4690
P1-C5	28	09/12/96	12.042	6.004	28.312	143.437	4.23×10^6	4450
P1-C6	28	09/12/96	12.015	6.053	28.776	142.441	4.17×10^6	4450
Average								**4530**
PC-C1	28	09/12/96	12.085	6.012	28.387	149.601	4.23×10^6	4370
PC-C2	28	09/12/96	12.044	6.025	28.510	149.965	4.21×10^6	4455
PC-C3	28	09/12/96	12.100	6.045	28.700	148.283	4.18×10^6	4425
Average								**4415**

Conversion table:

1 inch = 25.4 mm — 1 psi = 0.006895 Mpa — °F to °C: T(°C) = [T(°F) - 32]/1.8

1 pcf = 16.02 kg/cu m — 1 pcy = 0.5933 kg/cu m — 1 inch-pound = 0.1130 Nm

1 sq. in. = 645.2 sq.mm — 1 lb = 0.4536 kgf = 4.448 N

Table B4: First Crack Strength and Maximum Flexural Strength

Specimen #	Age (Days)	First Crack Load (lbs)	First Crack Deflection (inches)	First Crack Stress (psi)	Maximum Load (lbs)	Flexural Strength (psi)
P1-B1	7	3011	0.0012	550	3031	554
P1-B2	7	2641	0.0008	481	2832	516
P1-B4	7	2750	0.0005	486	3071	542
Average				**506**		**537**
P1-B5	28	3711	0.0006	641	3715	642
P1-B6	28	3686	0.0009	663	3926	706
P1-B7	28	3540	0.0006	621	3569	626
P1-B8	28	2859	0.0008	499	3370	589
Average				**606**		**641**
PC-B1	28	-	-	-	2907	521
PC-B2	28	-	-	-	3436	619
PC-B3	28	-	-	-	3164	549
PC-B4	28	-	-	-	3050	510
Average						**550**

P1-B3: Failed before testing

Conversion table:
1 inch = 25.4 mm
1 psi = 0.006895 Mpa
°F to °C: T(°C) = [T(°F) - 32]/1.8
1 pcf = 16.02 kg/cu m
1 pcy = 0.5933 kg/cu m
1 inch-pound = 0.1130 Nm
1 sq. in. = 645.2 sq mm
1 lb = 0.4536 kgf = 4.448 N

Table B5: Japanese Standard - Toughness & Equivalent Flexural Strength

Specimen #	Age (Days)	Toughness (Inch-lbs)	Equivalent Flexural Strength (psi)
P1-B1	7	120	274
P1-B2	7	133	303
P1-B4	7	119	264
Average		**124**	**280**
P1-B5	28	125	269
P1-B6	28	133	298
P1-B7	28	160	352
P1-B8	28	126	276
Average		**136**	**299**

Table B6: ASTM Toughness Indices

Specimen #	Age (Days)	First Crack Toughness (inch-lbs)	Toughness Indices			Toughness Ratios	
			I5	I10	I20	I10/I5	I20/I10
P1-B1	7	2.24	4.14	7.70	13.57	1.86	1.76
P1-B2	7	1.20	4.66	8.91	16.29	1.91	1.83
P1-B4	7	1.12	3.68	6.91	12.89	1.88	1.87
Average		**1.52**	**4.16**	**7.84**	**14.25**	**1.88**	**1.82**
P1-B5	28	1.72	3.54	6.54	12.00	1.85	1.84
P1-B6	28	2.54	3.67	6.67	11.50	1.82	1.72
P1-B7	28	1.53	3.76	7.04	13.08	1.88	1.86
P1-B8	28	1.64	4.18	7.92	14.50	1.89	1.83
Average		**1.86**	**3.79**	**7.04**	**12.77**	**1.86**	**1.81**

Conversion table:
1 inch = 25.4 mm   1 psi = 0.006895 Mpa   °F to °C: T(°C) = [T(°F) - 32]/1.8
1 pcf = 16.02 kg/cu m   1 pcy = 0.5933 kg/cu m   1 inch-pound = 0.1130 Nm
1 sq. in. = 645.2 sq mm   1 lb = 0.4536 kgf = 4.448 N

Table B7: Impact Strength

[Specimens made from the paving done on August 15, 1996 for Full-Depth Pavement on Highway 83 (North Bound Lane)]

Specimen #	Number of Blows	
	First Crack	Final Failure
P1-I1	22	188
P1-I2	21	193
P1-I3	29	246
P1-I4	21	328
P1-I5	61	261
P1-I6	37	244
P1-I7	35	178
P1-I8	26	253
P1-I9	75	457
P1-I10	162	523
P1-I11	17	242
P1-I12	19	422
P1-I13	75	275
P1-I14	56	477
P1-I15	46	318
P1-I16	29	218
P1-I17	17	194
P1-I18	36	211
P1-I19	38	473
P1-I20	29	247

Specimens made from the paving done on August 26 and 27, 1996 for Full-Depth Pavement on Highway 83 (South Bound Lane)

Table B8: Fresh Concrete Properties

Mixture #	Time	Ambient Temp.	Ambient Humidity	Conc. Temp. * 50°-90°	Unit Weight	Air Content * 6.5±1.5 %	Slump * 2" Max	Actual Fiber Content * 25 lbs/yd3
		(°F)	(%)	(°F)	(lb/cu ft)	(%)	Inches	(lb/cu yd)
P2	3:05	86	26	83.2	144.612	8.4	1-1/2	26.00
(8/26/96)	4:05	86	25	82.3	145.642	7.6	1	-
	4:55	85	20	82.1	145.642	7.6	1-1/2	23.23
	5:15	83	20	-	145.642	-	-	9.48**
	5:30	81	23	83.6	145.127	7.8	1-1/2	-
	6:30	74	27	81.7	145.642	8.6	2-3/4	-
P3	7:55	53	72	74.4	145.951	6.0	3/4	-
(8/27/96)	8:30	55	70	74.2	145.951	6.2	1	23.62
	9:30	62	65	77.2	145.127	5.9	3/4	23.78

* DOT Specification Range

** Concrete placed roughly around 317±10. Inspection of the concrete showed very few fibers. Therefore this test was taken, which confirmed our observation.

Table B9: Number of Specimens

Mixture #	Number of Specimens		
	Beams	Cylinders	Impact
P2	8	6	-
P3	8	6	-

Conversion table:

1 inch = 25.4 mm | 1 psi = 0.006895 Mpa | °F to °C: T(°C) = [T(°F) - 32]/1.8

1 pcf = 16.02 kg/cu m | 1 pcy = 0.5933 kg/cu m | 1 inch-pound = 0.1130 Nm

1 sq. in. = 645.2 sq mm | 1 lb = 0.4536 kgf = 4.448 N

Table B10: Compressive Strength

Specimen #	Age (days)	Date of Testing	Length (inches)	Diameter (inches)	Area (Sq. In.)	Unit Weight (pcf)	Static Modulus (psi)	Comp. Str. (psi)
P2-C1	7	09/02/96	12.212	6.009	28.359	143.451	4.23×10^6	3525
P2-C2	7	09/02/96	12.094	6.057	28.814	142.563	4.16×10^6	3560
P2-C3	7	09/02/96	12.093	6.042	28.672	141.412	4.19×10^6	3420
Average								**3500**
P2-C4	28	09/23/96	12.170	5.994	28.218	143.407	4.25×10^6	4075
P2-C5	28	09/23/96	12.163	6.005	28.321	142.968	4.24×10^6	4060
P2-C6	28	09/23/96	12.174	6.002	28.293	142.980	4.24×10^6	4085
Average								**4075**
P3-C1	7	09/03/96	12.071	5.989	28.171	144.825	4.26×10^6	3730
P3-C2	7	09/03/96	12.155	6.020	28.463	142.349	4.22×10^6	3690
P3-C3	7	09/03/96	12.050	5.986	28.143	142.674	4.26×10^6	3730
Average								**3715**
P3-C4	28	09/24/96	12.055	5.998	28.255	144.586	4.25×10^6	5240
P3-C5	28	09/24/96	12.120	6.008	28.350	143.328	4.23×10^6	4940
P3-C6	28	09/24/96	12.058	6.024	28.501	143.302	4.21×10^6	4980
Average								**5055**

Conversion table:
1 inch = 25.4 mm   1 psi = 0.006895 Mpa   °F to °C: T(°C) = [T(°F) - 32]/1.8
1 pcf = 16.02 kg/cu m   1 pcy = 0.5933 kg/cu m   1 inch-pound = 0.1130 Nm
1 sq. in. = 645.2 sq mm   1 lb = 0.4536 kgf = 4.448 N

Table B11: First Crack Strength and Maximum Flexural Strength

Specimen #	Age (Days)	First Crack Load (lbs)	First Crack Deflection (inches)	First Crack Stress (psi)	Maximum Load (lbs)	Flexural Strength (psi)
P2-B1	7	2133	0.0004	386	2153	390
P2-B2	7	2693	0.0005	481	2699	482
P2-B3	7	2900	0.0005	514	2907	515
P2-B4	7	3181	0.0012	573	3200	576
Average				**489**		**491**
P2-B5	28	2999	0.001	547	3002	548
P2-B6	28	3496	0.0008	639	3643	666
P2-B7	28	2301	0.0008	418	2797	508
P2-B8	28	3279	0.0008	595	3305	600
Average				**550**		**581**
P3-B1	7	3268	0.0007	591	3389	613
P3-B2	7	3138	0.0012	546	3237	563
P3-B4	7	3085	0.0008	549	3092	551
Average				**562**		**576**
P3-B5	28	3842	0.001	711	4088	756
P3-B6	28	4075	0.0012	746	4200	769
P3-B7	28	3909	0.0005	708	3914	709
P3-B8	28	4246	0.0008	776	4271	780
Average				**735**		**754**

P3-B3: Failed before testing

Conversion table:
1 inch = 25.4 mm 1 psi = 0.006895 Mpa °F to °C: T(°C) = [T(°F) - 32]/1.8
1 pcf = 16.02 kg/cu m 1 pcy = 0.5933 kg/cu m 1 inch-pound = 0.1130 Nm
1 sq. in. = 645.2 sq mm 1 lb = 0.4536 kgf = 4.448 N

Table B12: Japanese Standard - Toughness & Equivalent Flexural Strength

Specimen #	Age (Days)	Toughness (Inch-lbs)	Equivalent Flexural Strength (psi)
P2-B1	7	82	187
P2-B2	7	100	222
P2-B3	7	140	309
P2-B4	7	155	350
Average		**119**	**267**
P2-B5	28	121	276
P2-B6	28	214	490
P2-B7	28	105	237
P2-B8	28	149	338
Average		**147**	**335**
P3-B1	7	97	219
P3-B2	7	130	283
P3-B4	7	113	251
Average		**113**	**251**
P3-B5	28	177	409
P3-B6	28	157	360
P3-B7	28	182	411
P3-B8	28	169	385
Average		**171**	**391**

Conversion table:
1 inch = 25.4 mm 1 psi = 0.006895 Mpa °F to °C: T(°C) = [T(°F) - 32]/1.8
1 pcf = 16.02 kg/cu m 1 pcy = 0.5933 kg/cu m 1 inch-pound = 0.1130 Nm
1 sq. in. = 645.2 sq mm 1 lb = 0.4536 kgf = 4.448 N

Table B13: ASTM Toughness Indices

Specimen #	Age (Days)	First Crack Toughness (inch-lbs)	Toughness Indices			Toughness Ratios	
			I5	I10	I20	I10/I5	I20/I10
P2-B1	7	0.60	3.85	7.29	13.76	1.89	1.89
P2-B2	7	1.08	3.47	6.43	11.94	1.85	1.86
P2-B3	7	1.07	3.70	6.97	13.19	1.89	1.89
P2-B4	7	3.14	3.39	6.23	11.38	1.84	1.83
Average		**1.47**	**3.60**	**6.73**	**12.57**	**1.87**	**1.87**
P2-B5	28	2.57	3.29	5.97	10.76	1.82	1.80
P2-B6	28	2.17	3.65	6.84	12.81	1.87	1.87
P2-B7	28	1.23	4.55	8.77	16.41	1.93	1.87
P2-B8	28	2.09	3.49	6.48	12.00	1.85	1.85
Average		**2.02**	**3.74**	**7.02**	**13.00**	**1.87**	**1.85**
P3-B1	7	1.57	3.94	7.35	13.30	1.87	1.81
P3-B2	7	2.50	4.01	7.49	13.43	1.87	1.79
P3-B4	7	1.69	3.86	7.19	13.06	1.86	1.82
Average		**1.92**	**3.94**	**7.34**	**13.26**	**1.87**	**1.81**
P3-B5	28	3.13	4.03	7.53	13.52	1.87	1.80
P3-B6	28	3.38	3.90	7.24	12.96	1.86	1.79
P3-B7	28	1.47	3.63	6.82	12.84	1.88	1.88
P3-B8	28	2.27	3.96	7.53	14.16	1.90	1.88
Average		**2.56**	**3.88**	**7.28**	**13.37**	**1.88**	**1.84**

Conversion table:
1 inch = 25.4 mm
1 psi = 0.006895 Mpa
°F to °C: T(°C) = [T(°F) - 32]/1.8
1 pcf = 16.02 kg/cu m
1 pcy = 0.5933 kg/cu m
1 inch-pound = 0.1130 Nm
1 sq. in. = 645.2 sq mm
1 lb = 0.4536 kgf = 4.448 N

Table B14 CYLINDER COMPRESSIVE STRENGTH & ELASTIC MODULUS OF MIXTURE DOT - T5 & T6

Mixture #	Specimen #	Date Tested	Age (days)	Dimensions Length mm (in)	Dimensions Diameter mm (in)	Unit Weight kg/m^3 (pcf)	Static Modulus MPa (10^6 psi)	Compressive Strength MPa (psi)
DOT-T5 Cast 7/12/96	DOT-T5-C1	7/18/97	371	309.17 (12.172)	151.69 (5.972)	2167.6 (135.32)	29538 (4.284)	50.46 (7319)
	DOT-T5-C2	7/18/97	371	304.32 (11.981)	151.94 (5.982)	2211.1 (138.04)	29455 (4.272)	49.31 (7151)
	Average			306.74 (12.077)	151.82 (5.977)	2189.3 (136.68)	**29497 (4.278)**	**49.89 (7235)**
	S. D.			3.43046	0.17961	30.808	58.51	0.8191
	% C. V.			28.41	3.00	22.54	1367.60	0.01

Mixture #	Specimen #	Date Tested	Age (days)	Dimensions Length mm (in)	Dimensions Diameter mm (in)	Unit Weight kg/m^3 (pcf)	Static Modulus MPa (10^6 psi)	Compressive Strength MPa (psi)
DOT-T6 Cast 7/12/96	DOT-T6-C1	7/31/97	384	308.10 (12.130)	151.82 (5.977)	2260.1 (141.10)	32613 (4.730)	51.73 (7502)
	DOT-T6-C2	7/31/97	384	309.22 (12.174)	151.99 (5.984)	2271.4 (141.80)	33841 (4.908)	52.71 (7645)
	Average			308.66 (12.152)	151.90 (5.981)	2265.7 (141.45)	**33227 (4.819)**	**52.22 (7574)**
	S. D.			0.7903	0.1257	7.929	867.8	0.6972
	% C. V.			6.50	2.10	5.61	18008.70	0.01

Table B15 CYLINDER COMPRESSIVE STRENGTH & ELASTIC MODULUS OF MIXTURE DOT - P1 & W1

Mixture #	Specimen #	Date Tested	Age (days)	Dimensions Length mm (in)	Dimensions Diameter mm (in)	Unit Weight kg/m^3 (pcf)	Static Modulus MPa (10^6 psi)	Compressive Strength MPa (psi)
P1 Cast 8/15/96	P1-C7	7/31/97	350	307.49 (12.106)	153.64 (6.049)	2164.0 (135.10)	23981 (3.478)	31.67 (4593)
	P1-C8	7/31/97	350	308.97 (12.164)	152.27 (5.995)	2184.9 (136.40)	24498 (3.553)	35.42 (5137)
	Average			308.23 (12.135)	152.96 (6.022)	2174.4 (135.75)	**24239 (3.516)**	**33.54 (4865)**
	S. D.			1.042	0.970	14.724	365.7	2.652
	% C. V.			8.58	16.11	10.85	10401.44	0.05

Mixture #	Specimen #	Date Tested	Age (days)	Dimensions Length mm (in)	Dimensions Diameter mm (in)	Unit Weight kg/m^3 (pcf)	Static Modulus MPa (10^6 psi)	Compressive Strength MPa (psi)
W1 Cast 7/23/96	W1-C7	7/31/97	373	307.98 (12.125)	153.24 (6.033)	2108.0 (131.60)	21381 (3.101)	28.82 (4180)
	W1-C8	7/31/97	373	305.26 (12.018)	153.29 (6.035)	2101.6 (131.20)	20968 (3.041)	30.13 (4370)
	W1-C9	7/31/97	373	303.89 (11.964)	152.81 (6.016)	2181.7 (136.20)	23829 (3.456)	32.02 (4644)
	W1-C10	7/31/97	373	305.00 (12.008)	153.87 (6.058)	2143.2 (133.80)	22043 (3.197)	34.21 (4961)
	W1-C11	7/31/97	373	304.77 (11.999)	152.40 (6.000)	2136.8 (133.40)	22981 (3.333)	34.39 (4987)
	W1-C12	7/31/97	373	307.14 (12.092)	152.81 (6.016)	2149.6 (134.20)	24560 (3.562)	31.05 (4503)
	Average			305.67 (12.034)	153.07 (6.026)	2136.8 (133.40)	**22627 (3.282)**	**31.77 (4608)**
	S. D.			1.553	0.5117	29.291	1410.8	2.224
	% C. V.			12.91	8.49	21.96	42991.68	0.05

Table B16 STATIC FLEXURAL STRENGTH OF MIXTURE DOT - T5 & T6

Mixture #	Specimen #	Date Tested	Age (days)	Maximum Load kg (lbs)	Dimensions at Failure Breadth mm (in)	Dimensions at Failure Depth mm (in)	Modulus of Rupture MPa (psi)
DOT-T5 Cast 7/12/96	DOT-T5-B7	7/18/97	371	2311 (5101)	104.1 (4.097)	104.1 (4.098)	6.14 (890)
	DOT-T5-B21	7/18/97	371	2239 (4943)	104.1 (4.100)	100.3 (3.950)	6.40 (927)
	DOT-T5-B23	7/18/97	371	2377 (5247)	104.5 (4.114)	107.2 (4.219)	5.93 (860)
	Average			2309 (5097)	104.2 (4.104)	103.9 (4.089)	**6.16 (892)**
	S. D.			68.9	0.2	3.5	0.2
	% C. V.			2.98	0.22	3.33	3.76

Mixture #	Specimen #	Date Tested	Age (days)	Maximum Load kg (lbs)	Dimensions at Failure Breadth mm (in)	Dimensions at Failure Depth mm (in)	Modulus of Rupture MPa (psi)
DOT-T6 Cast 7/12/96	DOT-T6-B10	7/30/97	383	2489 (5494)	104.9 (4.130)	104.1 (4.097)	6.58 (953)
	DOT-T6-B12	7/30/97	383	2585 (5707)	103.7 (4.082)	101.7 (4.005)	7.17 (1039)
	DOT-T6-B14	7/30/97	383	2591 (5720)	104.2 (4.104)	103.2 (4.062)	7.03 (1019)
	Average			2555 (5640)	104.3 (4.105)	103.0 (4.055)	**6.93 (1004)**
	S. D.			57.5	0.6	1.2	0.3
	% C. V.			2.25	0.585	1.145	4.48

Table B17 STATIC FLEXURAL STRENGTH OF MIXTURE DOT - P1 & W1

Mixture #	Specimen #	Date Tested	Age (days)	Maximum Load kg (lbs)	Dimensions at Failure Breadth mm (in)	Dimensions at Failure Depth mm (in)	Modulus of Rupture MPa (psi)
P1 Cast 8/15/96	P1-B9	8/1/97	385	2079 (4589)	106.5 (4.191)	104.1 (4.097)	5.40 (783)
	P1-B15	8/1/97	385	2161 (4771)	105.4 (4.149)	102.9 (4.052)	5.80 (840)
	P1-B21	8/1/97	385	2312 (5104)	102.4 (4.032)	102.2 (4.025)	6.47 (938)
	Average			2184 (4821)	104.7 (4.124)	103.1 (4.058)	**5.89 (854)**
	S. D.			118.3	2.1	0.9	0.5
	% C. V.			5.42	1.998	0.896	9.18

Mixture #	Specimen #	Date Tested	Age (days)	Maximum Load kg (lbs)	Dimensions at Failure Breadth mm (in)	Dimensions at Failure Depth mm (in)	Modulus of Rupture MPa (psi)
W1 Cast 7/23/96	W1-B11	8/1/97	385	2545 (5619)	104.1 (4.098)	103.2 (4.063)	6.88 (997)
	W1-B19	8/1/97	385	2047 (4519)	102.2 (4.025)	102.9 (4.052)	5.66 (821)
	W1-B24	8/1/97	385	2216 (4891)	101.2 (3.986)	102.2 (4.025)	6.27 (909)
	W1-B28	8/1/97	385	2730 (6027)	103.3 (4.067)	103.7 (4.081)	7.37 (1068)
	Average			2385 (5264)	102.7 (4.044)	103.0 (4.055)	**6.55 (949)**
	S. D.			309.7	1.2	0.6	0.7
	% C. V.			12.99	1.209	0.578	11.29

Table B18 FATIGUE STRENGTH CHARACTERISTICS

Mixture	f_r MPa (psi)	f_{max} MPa (psi)	f_c' MPa (psi)	f_{rn} MPa (psi)	$f_{max,n}$ MPa (psi)	$E.L_1$ (%)	$E.L_2$ (%)	$N.E.L_1$ (%)	$N.E.L_2$ (%)
Plain	5.10 (739)	3.45 (501)	42.96 (6230)	4.79 (694)	3.25 (471)	68	68	68	68
DOT-T5	6.15 (892)	3.49 (506)	49.89 (7235)	5.36 (778)	3.04 (441)	57	68	57	64
DOT-T6	6.92 (1004)	4.18 (606)	52.22 (7574)	5.90 (856)	3.56 (516)	60	82	60	74
P1	5.89 (854)	4.15 (602)	33.54 (4865)	6.26 (908)	4.41 (640)	70	81	70	92
W1	6.54 (949)	4.21 (611)	31.77 (4608)	7.15 (1037)	4.60 (668)	64	83	64	96

f_r : Average Static Flexural Stress
f_{max} : Flexural Fatigue Stress (Maximum)
f_c' : Average Cylinder Compressive Strength
f_{rn} : Normalized Average Static Flexural Stress
$f_{max,n}$: Normalized Flexural Fatigue Stress (Maximum)
$E.L_1$: Endurance Limit based on its own Modulus of Rupture (f_{max}/f_r)
$E.L_2$: Endurance Limit based on Plain Concrete (f_{max}/f_{rc})
$N.E.L_1$: Normalized Endurance Limit based on its own Modulus of Rupture ($f_{max,n}/f_{rn}$)
$N.E.L_2$: Normalized Endurance Limit based on Plain Concrete ($f_{max,n}/f_{rnc}$)

Table B19 FATIGUE TEST RESULTS FOR MIXTURE DOT-T5

Mixture #	Specimen #	Date Tested	Age (days)	Maximum Load kg (lbs)	No. of cycles to failure	Dimensions at Failure		f_f		f_f / f_r	Remarks
						Breadth mm (in)	Depth mm (in)	MPa	(psi)		
	DOT-T5-B10	7/27/97	380	1081 (2380)	2,287,600	104.7 (4.124)	101.2 (3.984)	3.01	(436)	0.49	Did not fail
DOT-T5	DOT-T5-B13	7/24/97	377	1082 (2380)	2,014,200	104.3 (4.106)	101.0 (3.976)	3.03	(440)	0.49	Did not fail
	DOT-T5-B3	7/25/97	378	1083 (2380)	2,001,900	105.1 (4.136)	103.2 (4.064)	2.88	(418)	0.47	Did not fail
	DOT-T5-B20	7/21/97	374	1189 (2620)	2,682,700	105.2 (4.143)	101.6 (4.001)	3.27	(474)	0.53	Did not fail
Cast	DOT-T5-B12	7/20/97	373	1190 (2620)	2,001,000	105.6 (4.157)	100.1 (3.940)	3.36	(487)	0.55	Noticed crack at 8:00 a.m. on 7/21/97 that was 3 mm wide and 3.5 inched long at 1,146,200 cycles. Specimen did not fail.
7/12/96	DOT-T5-B5	7/24/97	377	1191 (2620)	313,500	102.5 (4.036)	102.2 (4.024)	3.32	(481)	0.54	Very few fibers in failure plane
	DOT-T5-B2	7/24/97	377	1192 (2620)	123,200	104.0 (4.095)	101.2 (3.984)	3.34	(484)	0.54	
	DOT-T5-B16	7/28/97	381	1294 (2850)	2,955,800	103.4 (4.072)	107.0 (4.212)	3.26	(473)	0.53	Crack noticed at 800 cycles. Noticed that crack had widened to 0.4 inches at 1,169,000 cycles. Crack widened to 0.5 inches at 1,656,600 cycles.
	DOT-T5-B19	7/18/97	371	1295 (2850)	2,015,500	103.0 (4.055)	103.7 (4.083)	3.49	(506)	0.57	Did not fail
	DOT-T5-B6	7/19/97	372	1296 (2850)	467,300	106.2 (4.181)	102.2 (4.022)	3.49	(506)	0.57	
	DOT-T5-B4	8/9/97	393	1296 (2854)	139,000	104.6 (4.118)	100.8 (3.970)	3.64	(528)	0.59	
	DOT-T5-B9	8/7/97	391	1296 (2854)	2,000,000	106.7 (4.202)	104.8 (4.125)	3.30	(479)	0.54	Crack noticed at 66,000 cycles 0.08 mm wide and 3" long
	DOT-T5-B8	7/18/97	371	1297 (2850)	188,700	104.1 (4.098)	102.2 (4.022)	3.56	(516)	0.58	
	DOT-T5-B17	7/20/97	373	1298 (2850)	2,200	104.7 (4.122)	103.9 (4.092)	3.42	(496)	0.56	
	DOT-T5-B18	8/11/97	395	1404 (3092)	2,100	106.0 (4.173)	103.2 (4.062)	3.72	(539)	0.60	
	DOT-T5-B15	7/18/97	371	1512 (3330)	4,800	102.9 (4.050)	102.0 (4.016)	4.22	(612)	0.69	
	DOT-T5-B22	7/23/97	376	1621 (3570)	1,667,700	104.7 (4.122)	104.1 (4.100)	4.26	(618)	0.69	Crack developed at 1300 cycles. Beam broke and machine did not stop. Number of cycles is not accurate.
	DOT-T5-B1	7/18/97	371	1728 (3806)	4,200	104.4 (4.112)	102.5 (4.037)	4.70	(682)	0.76	
	DOT-T5-B11	7/23/97	376	1836 (4045)	45,500	105.0 (4.135)	103.7 (4.081)	4.86	(705)	0.79	Crack appeared at 3000 cycles. 8000 cycles crack reached the top & still holds load. Not fully broken, but deflection stopped machine.
	DOT-T5-B14	9/30/97	445	1404 (3092)	523,100	101.8 (4.011)	101.2 (3.984)	4.02	(583)	0.65	Crack observed at 1900 cycles

Table B20 FATIGUE TEST RESULTS FOR MIXTURE DOT - T6

Mixture #	Specimen #	Date Tested	Age (days)	Maximum Load kg (lbs)	No. of cycles to failure	Dimensions at Failure: Breadth mm (in)	Dimensions at Failure: Depth mm (in)	f_f MPa	f_f (psi)	f_f / f_r	Remarks
DOT-T6 Cast 7/12/96	DOT-T6-B15	8/11/97	395	1216 (2678)	44,100	103.9 (4.091)	103.8 (4.085)	3.25	(471)	0.47	Few fibers in cross-section
	DOT-T6-B13	8/13/97	397	1337 (2945)	2,000,100	103.1 (4.059)	103.0 (4.055)	3.65	(529)	0.53	Crack noticed at 1772000 cycles
	DOT-T6-B17	8/11/97	395	1337 (2945)	2,000,100	105.3 (4.144)	103.5 (4.073)	3.54	(514)	0.51	Crack noticed at 273,700 cycles
	DOT-T6-B20	8/14/97	398	1459 (3213)	1,499,000	104.0 (4.093)	102.6 (4.056)	3.95	(573)	0.57	Crack noticed at 236,800 cycles
	DOT-T6-B7	8/15/97	399	1459 (3213)	2,800	103.1 (4.059)	102.8 (4.046)	4.00	(580)	0.58	
	DOT-T6-B1	8/1/97	385	1460 (3215)	2,000,000	102.2 (4.025)	102.6 (4.040)	4.05	(587)	0.58	Did not fail. Crack formed at 2400 cycles & at 14600 cycles it was 7 mm wide and 3.9 inches high.
	DOT-T6-B2	8/3/97	387	1461 (3215)	695,800	101.0 (3.975)	103.0 (4.055)	4.07	(590)	0.59	Crack appeared at 5000 cycles. Crack width was 3 mm and 3.8 inches high.
	DOT-T6-B16	7/31/97	384	1462 (3215)	2,000,000	102.6 (4.041)	103.7 (4.083)	3.95	(573)	0.57	Crack observed at 1,000,000 cycles. Crack was 0.4 mm wide and 2.75 inches high. The crack remained the same upto 2,000,000 cycles.
	DOT-T6-B18	9/10/97	425	1580 (3481)	6,100	104.9 (4.130)	103.8 (4.085)	4.18	(606)	0.60	Crack developed at 1800 cycles.
	DOT-T6-B21	9/10/97	425	1580 (3481)	1,700,300	104.1 (4.097)	102.8 (4.049)	4.29	(622)	0.62	Crack developed at 2600 cycles
	DOT-T6-B8	9/11/97	426	1702 (3749)	4,200	101.1 (3.980)	103.9 (4.089)	4.66	(676)	0.67	Crack developed at 2100 cycles.
	DOT-T6-B3	7/31/97	384	1823 (4015)	3,200	103.9 (4.090)	104.5 (4.114)	4.80	(696)	0.69	Crack developed at 1700 cycles.
	DOT-T6-B9	7/30/97	383	1945 (4285)	2,000,000	102.5 (4.037)	104.5 (4.115)	5.19	(752)	0.75	Crack appeared at 3000 cycles. Did not fail.
	DOT-T6-B19	7/30/97	383	2066 (4550)	1,900	103.8 (4.088)	103.0 (4.057)	5.59	(811)	0.81	specimen failed at 1900 cycles
	DOT-T6-B11	10/10/97	456	1824 (4016)	600	102.2 (4.022)	102.6 (4.04)	5.06	(734)	0.73	Specimen failed at 600 cycles
	DOT-T6-B22	9/16/97	431	1824 (4016)	1,100	103.3 (4.068)	102.7 (4.046)	4.99	(724)	0.72	Specimen failed at 1100 cycles
	DOT-T6-B23	9/16/97	431	1824 (4016)	1,100	103.7 (4.086)	104.5 (4.115)	4.81	(697)	0.69	Crack developed at 400 cycles
	DOT-T6-B4	10/3/97	449	1702 (3749)	2,000,100	102.2 (4.022)	102.7 (4.046)	4.71	(683)	0.68	Crack developed at 1100 cycles
	DOT-T6-B5	9/30/97	445	1580 (3481)	2,000,100	101.9 (4.012)	103 (4.055)	4.36	(633)	0.63	Crack developed at 5400 cycles
	DOT-T6-B6	10/2/97	448	1580 (3481)	2,000,100	101.7 (4.006)	102.6 (4.042)	4.40	(638)	0.64	Crack developed at 2000 cycles

Table B21 FATIGUE TEST RESULTS FOR MIXTURE DOT - P1

Mixture #	Specimen #	Date Tested	Age (days)	Maximum Load kg (lbs)	No. of cycles to failure	Dimensions at Failure Breadth mm (in)	Depth mm (in)	f_f MPa	(psi)	f_f / f_r	Remarks
P1	P1-B24	8/18/97	368	1034 (2278)	2,000,000	105.0 (4.134)	102.3 (4.027)	2.81	(408)	0.477752	Did not fail
	P1-B17	8/21/97	371	1137 (2505)	2,000,100	104.8 (4.126)	104.5 (4.115)	2.96	(430)	0.50	Crack noticed at 1,200,000 cycles.
	P1-B30	8/20/97	370	1137 (2505)	2,000,000	105.9 (4.168)	103.3 (4.067)	3.01	(436)	0.51	Did not fail
	P1-B19	8/26/97	376	1241 (2733)	220,200	104.0 (4.095)	105.8 (4.164)	3.19	(462)	0.54	Crack noticed at 55,000 cycles
	P1-B33	8/25/97	375	1241 (2733)	2,000,000	106.2 (4.180)	105.0 (4.135)	3.16	(459)	0.54	Did not fail
Cast	P1-B16	8/27/97	377	1344 (2961)	2,000,000	106.1 (4.178)	103.3 (4.065)	3.55	(515)	0.60	Crack noticed at 13,000 cycles. At 1,201,700 cycles machine stopped and restarted.
8/15/96	P1-B23	9/11/97	392	1448 (3189)	12,100	104.6 (4.117)	103.9 (4.090)	3.83	(556)	0.65	Crack noticed at 1700 cycles
	P1-B13	8/6/97	356	1551 (3416)	2,000,000	105.2 (4.142)	102.1 (4.021)	4.22	(612)	0.72	2200 cycles a 0.10 mm wide crack appeared
	P1-B18	8/4/97	354	1654 (3644)	2,000,000	105.3 (4.146)	103.4 (4.070)	4.39	(637)	0.75	At 1500 cycles crack developed. The crack was 2 mm wide and 3.8 inches high at 7500 cycles. At 1,058,200 cycles the crack width was 5 mm. At 1,058,200 a power failure stopped the machine. We restarted it on 8/6/97.
	P1-B25	8/6/97	356	1654 (3644)	2,900	103.8 (4.085)	102.9 (4.051)	4.50	(652)	0.76	Crack developed at 1200 cycles - 3 mm wide. Very few fibers in beam.
	P1-B31	8/1/97	351	1757 (3870)	2,700	104.5 (4.115)	103.3 (4.066)	4.71	(683)	0.80	Cracked at 1400 cycles.
	P1-B10	10/4/97	450	1344 (2961)	2,000,100	107.6 (4.24)	102.3 (4.026)	3.56	(517)	0.61	Crack size = 1mm
	P1-B11	10/13/97	459	1551.3 (3416)	2,000,100	106.7 (4.202)	102.3 (4.026)	4.15	(602)	0.70	Specimen did not fail
	P1-B12	10/14/97	460	1654.8 (3644)	2,000,100	102.8 (4.048)	101.7 (4.006)	4.64	(673)	0.79	
	P1-B14	10/11/97	457	1862 (4100)	700	104.1 (4.098)	103 (4.055)	5.03	(730)	0.85	
	P1-B20	9/14/97	395	1551.3 (3416)	4,000	104.2 (4.102)	105.5 (4.154)	3.99	(579)	0.68	Crack observed at 1600 cycles
	P1-B22	9/14/97	395	1551.3 (3416)	2,000,100	103 (4.055)	103.9 (4.092)	4.16	(604)	0.71	Crack observed at 2600 cycles. Specimen did not fail at 2000100 cycles
	P1-B26	10/6/97	452	1344 (2961)	2,000,100	106.1 (4.179)	104.3 (4.107)	3.47	(503)	0.59	No crack was observed
	P1-B27	10/6/97	452	1344 (2961)	42,600	106.2 (4.180)	102 (4.014)	3.64	(528)	0.62	Crack observed at 1400 cycles
	P1-B28	10/7/97	453	1448 (3189)	2,000,100	104.1 (4.099)	101.8 (4.01)	4.01	(581)	0.68	Crack observed at 1600 cycles
	P1-B29	10/9/97	455	1448 (3189)	2,000,100	106.2 (4.180)	102.7 (4.046)	3.85	(559)	0.65	Crack observed at 1300 cycles
	P1-B32	10/11/97	457	1758 (3872)	3600	105 (4.130)	103.1 (4.060)	4.71	(683)	0.80	Crack observed at 1100 cycles
	P1-B34	10/11/97	457	1654 (3644)	37,000	104.4 (4.112)	102.1 (4.020)	4.54	(658)	0.77	Crack observed at 1300 cycles
	P1-B35	10/10/97	456	1551.3 (3416)	2,000,100	105.3 (4.148)	103 (4.052)	4.15	(602)	0.70	Crack observed at 1100 cycles

Table B22 FATIGUE TEST RESULTS FOR MIXTURE DOT - W1

Mixture #	Specimen #	Date Tested	Age (days)	Maximum Load kg (lbs)	No. of cycles to failure	Dimensions at Failure Breadth mm (in)	Dimensions at Failure Depth mm (in)	f_f MPa	f_f (psi)	f_f / f_r	Remarks
W1 Cast 7/23/96	W1-B16	8/29/97	402	1149 (2531)	2,000,000	105.4 (4.125)	102.1 (4.019)	3.14	(456)	0.48	Did not fail
	W1-B18	9/2/97	406	1264 (2784)	2,000,100	101.7 (4.004)	101.8 (4.009)	3.58	(519)	0.55	Crack developed at 4500 cycles
	W1-B31	9/4/97	408	1264 (2784)	1,900	101.0 (3.975)	101.8 (4.006)	3.61	(524)	0.55	Crack developed at 1500 cycles
	W1-B36	9/4/97	408	1379 (3037)	2,000,100	102.0 (4.015)	103.0 (4.055)	3.81	(552)	0.58	Crack noticed at 1500 cycles
	W1-B38	9/7/97	411	1379 (3037)	2,000,000	103.3 (4.068)	102.8 (4.049)	3.76	(546)	0.58	Crack developed at 1800 cycles
	W1-B15	9/8/97	412	1494 (3290)	1,816,700	103.6 (4.077)	101.1 (3.980)	4.21	(611)	0.64	Crack developed at 900 cycles
	W1-B20	9/9/97	413	1494 (3290)	5,300	103.7 (4.083)	103.1 (4.058)	4.05	(587)	0.62	Crack developed at 5300 cycles
	W1-B26	9/11/97	415	1609 (3543)	1,600	102.7 (4.042)	102.9 (4.050)	4.42	(641)	0.68	Crack developed at 1100 cycles
	W1-B9	10/11/97	457	1609 (3543)	3,100	102.1 (4.021)	102 (4.015)	4.52	(656)	0.69	Crack developed at 1000 cycles
	W1-B10	10/11/97	457	1724 (3796)	2,000,000	100.1 (3.940)	101.9 (4.014)	4.95	(718)	0.76	Crack developed at 1000 cycles
	W1-B12	9/29/97	444	1609 (3543)	904,700	102.8 (4.048)	102.3 (4.030)	4.46	(647)	0.68	Crack developed at 1700 cycles
	W1-B13	9/28/97	443	1609 (3543)	2,000,000	103.6 (4.078)	103.2 (4.066)	4.35	(631)	0.66	Crack developed at 1500 cycles
	W1-B14	9/28/97	443	1724 (3796)	1,300	103 (4.056)	102.2 (4.025)	4.78	(693)	0.73	Specimen Failed at 1300 cycles
	W1-B17	9/28/97	443	1839 (4049)	49,400	103.1 (4.058)	103 (4.052)	5.03	(729)	0.77	Crack developed at 900 cycles
	W1-B21	9/16/97	420	1724 (3796)	3,200	101 (3.98)	100.4 (3.955)	5.05	(732)	0.77	Crack developed at 700 cycles
	W1-B22	9/15/97	419	1724 (3796)	2,000,100	101.7 (4.006)	101.5 (3.998)	4.90	(711)	0.75	Crack developed at 1300 cycles
	W1-B23	9/27/97	442	1954 (4302)	2,000,100	101.8 (4.008)	102 (4.016)	5.51	(799)	0.84	Crack developed at 1100 cycles
	W1-B25	9/27/97	442	1954 (4302)	1,400	102.7 (4.043)	101.5 (3.998)	5.51	(799)	0.84	Crack developed at 800 cycles
	W1-B27	10/13/97	459	1954 (4302)	1,300	101.5 (3.996)	104.2 (4.103)	5.29	(767)	0.81	Crack developed at 100 cycles
	W1-B29	10/15/97	461	1609 (3543)	2,000,100	104.4 (4.110)	104.3 (4.106)	4.23	(614)	0.65	Crack developed at 900 cycles
	W1-B30	2/7/98	545	1264 (2784)	2,000,100	105.7 (4.162)	102.3 (4.026)	3.41	(495)	0.52	Crack developed at 16000 cycles
	W1-B32	10/13/97	459	1724 (3796)	1,700	102 (4.012)	101.4 (3.995)	4.90	(711)	0.75	Crack developed at 400 cycles
	W1-B33	2/9/98	547	1609 (3543)	5,600	103.8 (4.090)	103.6 (4.080)	4.31	(624)	0.66	Crack developed at 3800 cycles
	W1-B34	2/8/98	546	1494 (3290)	2,000,100	103 (4.056)	102.8 (4.05)	4.09	(593)	0.63	Crack developed at 2600 cycles. Specimen was tested again after two million cycles at 75 % dynamic load.
	W1-B34A	2/9/98	547	1494 (3290)	1,142,600	103 (4.056)	102.8 (4.05)	4.72	(685)	0.72	Specimen failed at 1,142,600 cycles
	W1-B35	2/9/98	547	1609 (3543)	8,100	105.3 (4.146)	102.7 (4.044)	4.32	(627)	0.66	Crack developed at 2100 cycles
	W1-B37	2/9/98	547	1724 (3796)	14,700	104.5 (4.116)	103.2 (4.062)	4.62	(671)	0.71	Crack developed at 3300 cycles

TABLE B23 STATIC FLEXURAL STRENGTH AFTER FATIGUE TEST OF MIXTURE DOT - T6 & T5

Mixture Designation	Specimen #	Maximum Load		Dimensions				f_{rA}	
				Breadth		Depth			
		kgs	(lbs)	mm	(in)	mm	(in)	Mpa	(psi)
DOT - T6	DOT-T6-B6	2292	(5052)	102.2	(4.022)	102.8	(4.046)	6.35	(921)
	DOT-T6-B5	3220	(7098)	101.9	(4.012)	103.0	(4.055)	8.91	(1291)
	DOT-T6-B4	2947	(6496)	102.2	(4.022)	102.8	(4.046)	8.17	(1184)
Average		2819	(6215)	102.1	(4.019)	102.8	(4.049)	**7.81**	**(1132)**

Mixture Designation	Specimen #	Maximum Load		Dimensions				f_{rA}	
				Breadth		Depth			
		kgs	(lbs)	mm	(in)	mm	(in)	Mpa	(psi)
DOT - T5	DOT-T5-B10	2635	(5810)	104.7	(4.124)	101.2	(3.984)	7.35	(1065)
	DOT-T5-B20	2626	(5789)	105.2	(4.143)	101.6	(4.001)	7.23	(1047)
	DOT-T5-B13	2552	(5627)	104.3	(4.106)	101.0	(3.976)	7.18	(1040)
	DOT-T5-B19	2571	(5667)	103.0	(4.055)	103.7	(4.083)	6.94	(1006)
	D0T-T5-B3	3740	(8246)	105.1	(4.136)	103.2	(4.064)	10.00	(1449)
Average		2825	(6228)	104.5	(4.113)	102.1	(4.022)	**7.74**	**(1121)**

TABLE B24 STATIC FLEXURAL STRENGTH AFTER FATIGUE TEST OF MIXTURE P1 & W1

Mixture Designation	Specimen #	Maximum Load		Dimensions				f_{rA}	
				Breadth		Depth			
		kgs	(lbs)	mm	(in)	mm	(in)	Mpa	(psi)
P1	P1-B33	3052	(6728)	106.2	(4.18)	105.0	(4.135)	7.79	(1130)
	P1-B28	1944	(4285)	104.1	(4.099)	101.9	(4.01)	5.38	(780)
	P1-B24	2289	(5046)	105.0	(4.134)	102.3	(4.027)	6.23	(903)
	P1-B22	1416	(3121)	103.0	(4.055)	103.9	(4.092)	3.81	(552)
	P1-B30	3577	(7887)	105.9	(4.168)	103.3	(4.067)	9.47	(1373)
	P1-B26	3063	(6753)	106.1	(4.179)	104.3	(4.107)	7.93	(1150)
	P1-B35	3599	(7935)	105.4	(4.148)	102.9	(4.052)	9.65	(1398)
	P1-B29	2640	(5821)	106.2	(4.18)	102.8	(4.046)	7.04	(1021)
	P1-B10	2186	(4820)	107.7	(4.24)	102.3	(4.026)	5.81	(842)
	P1-B12	3468	(7646)	102.8	(4.048)	101.8	(4.006)	9.75	(1412)
	P1-B11	3742	(8249)	106.7	(4.202)	102.3	(4.026)	10.03	(1453)
Average		2816	(6208)	105.4	(4.148)	103.0	(4.054)	**7.54**	**(1092)**

Mixture Designation	Specimen #	Maximum Load		Dimensions				f_{rA}	
				Breadth		Depth			
		kgs	(lbs)	mm	(in)	mm	(in)	Mpa	(psi)
W1	W1-B29	2244	(4946)	104.4	(4.11)	104.3	(4.106)	5.91	(857)
	W1-B36	1581	(3486)	102.0	(4.015)	103.0	(4.055)	4.37	(634)
	W1-B18	1672	(3686)	101.7	(4.004)	101.8	(4.009)	4.74	(687)
	W1-B38	1691	(3729)	103.3	(4.068)	102.8	(4.049)	4.63	(671)
	W1-B16	3266	(7201)	104.8	(4.125)	102.1	(4.019)	8.95	(1297)
	W1-B10	2844	(6269)	100.1	(3.94)	102.0	(4.014)	8.18	(1185)
	W1-B13	2718	(5991)	103.6	(4.078)	103.3	(4.066)	7.36	(1066)
	WI-B30	2285	(5037)	105.7	(4.162)	102.3	(4.026)	6.18	(896)
	W1-B23	1328	(2927)	101.8	(4.008)	102.0	(4.016)	3.75	(543)
	W1-B22	2865	(6315)	101.8	(4.006)	101.5	(3.998)	8.17	(1184)
Average		2249	(4959)	102.9	(4.052)	102.5	(4.036)	**6.22**	**(902)**

Fig B1: Comparison of Slump measured during Qulaity Control Tests

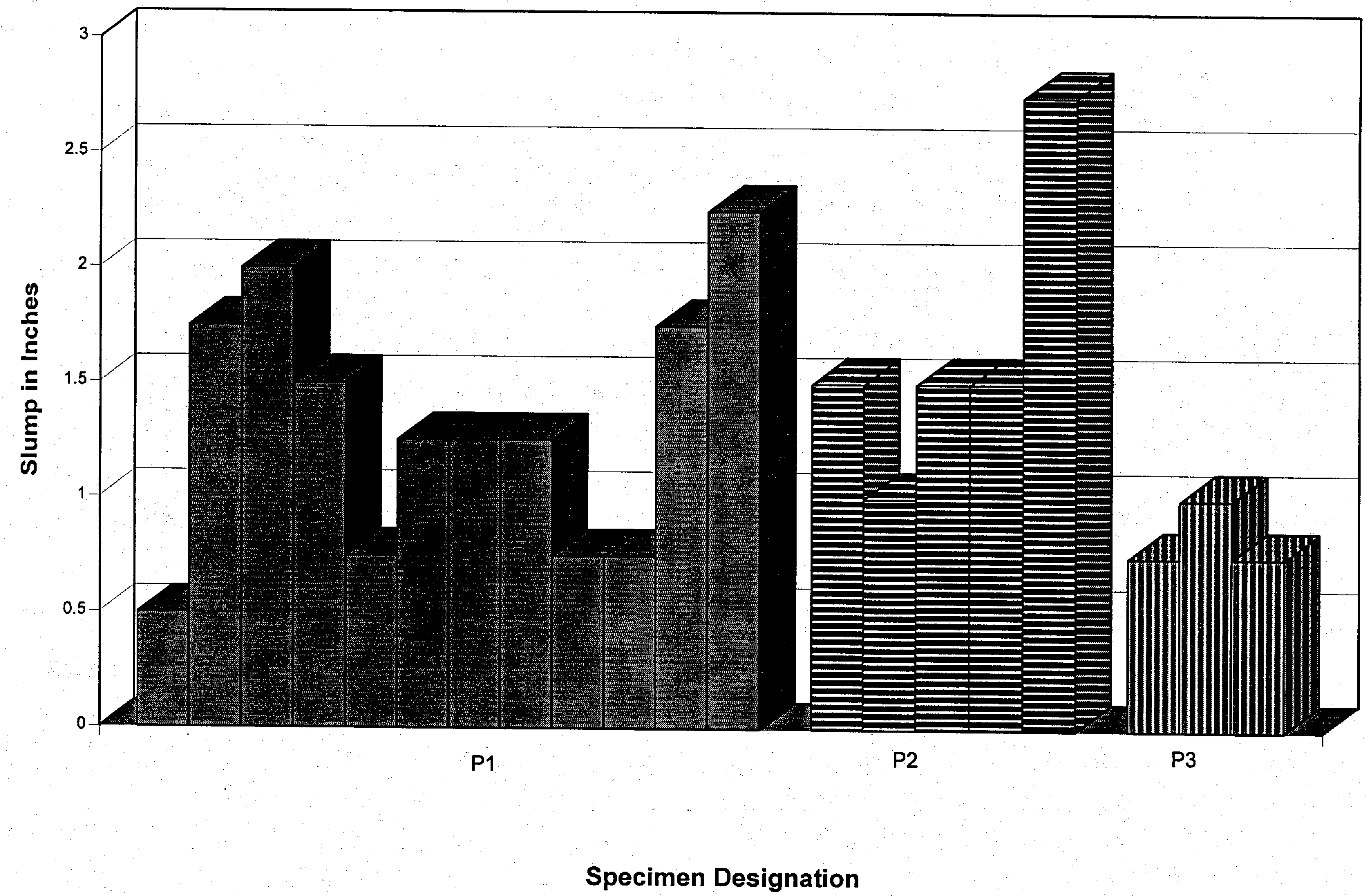


Fig B2: Comparison of Air Content measured during Qulaity Control Tests

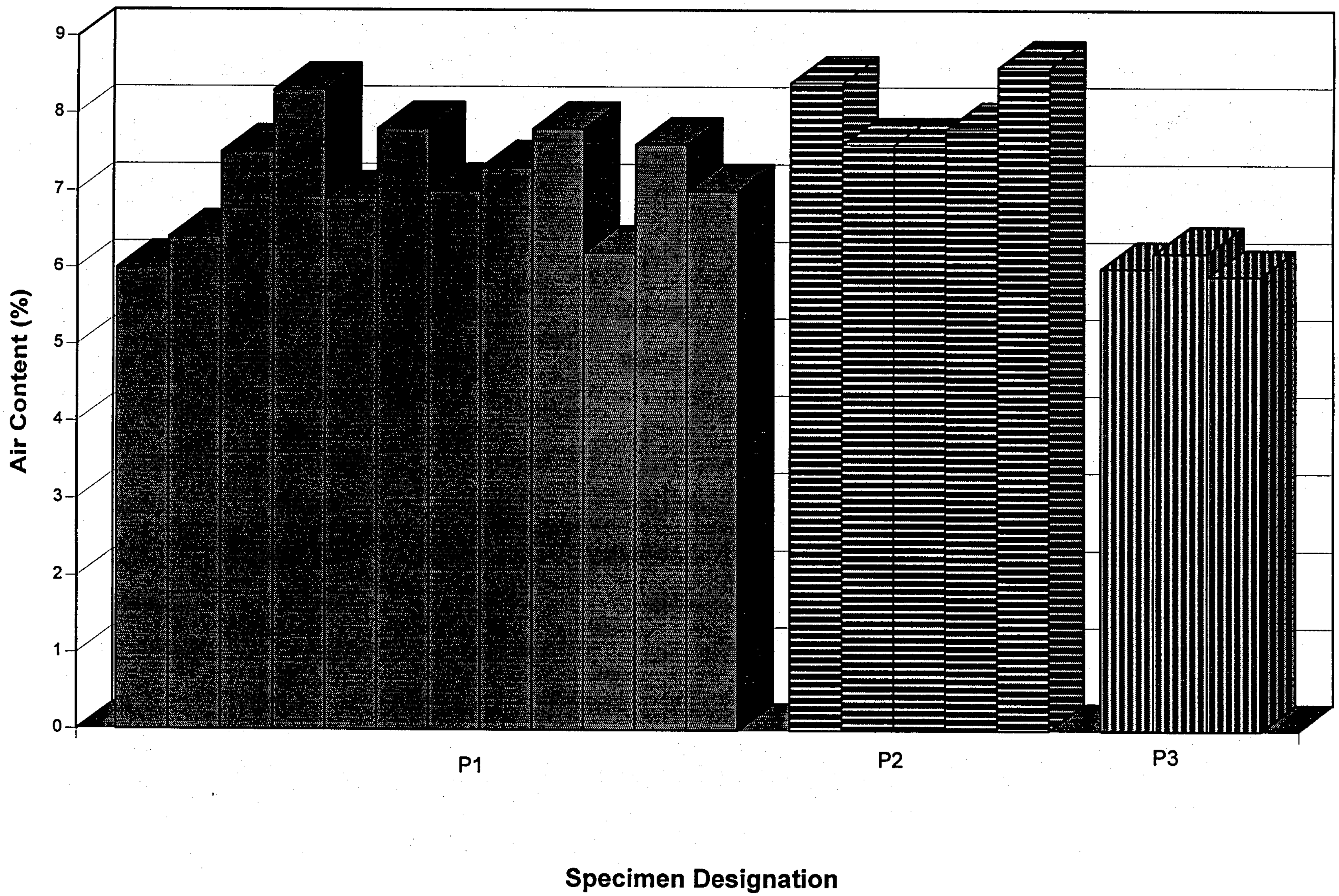


Fig B3: Comparison of Compressive Strength for Different Sections

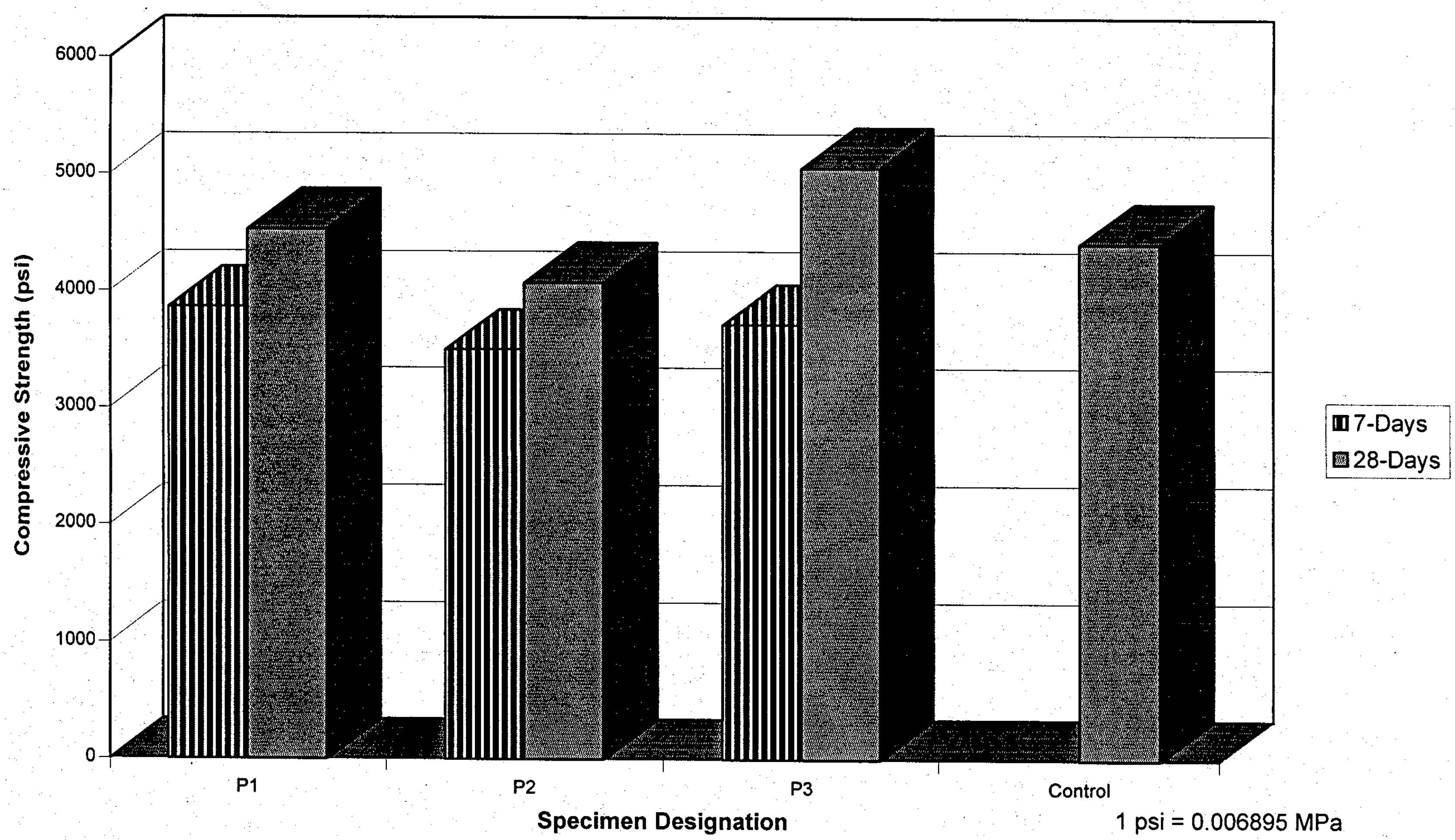


Fig B4: Comparison of First Crack Stress for Different Sections

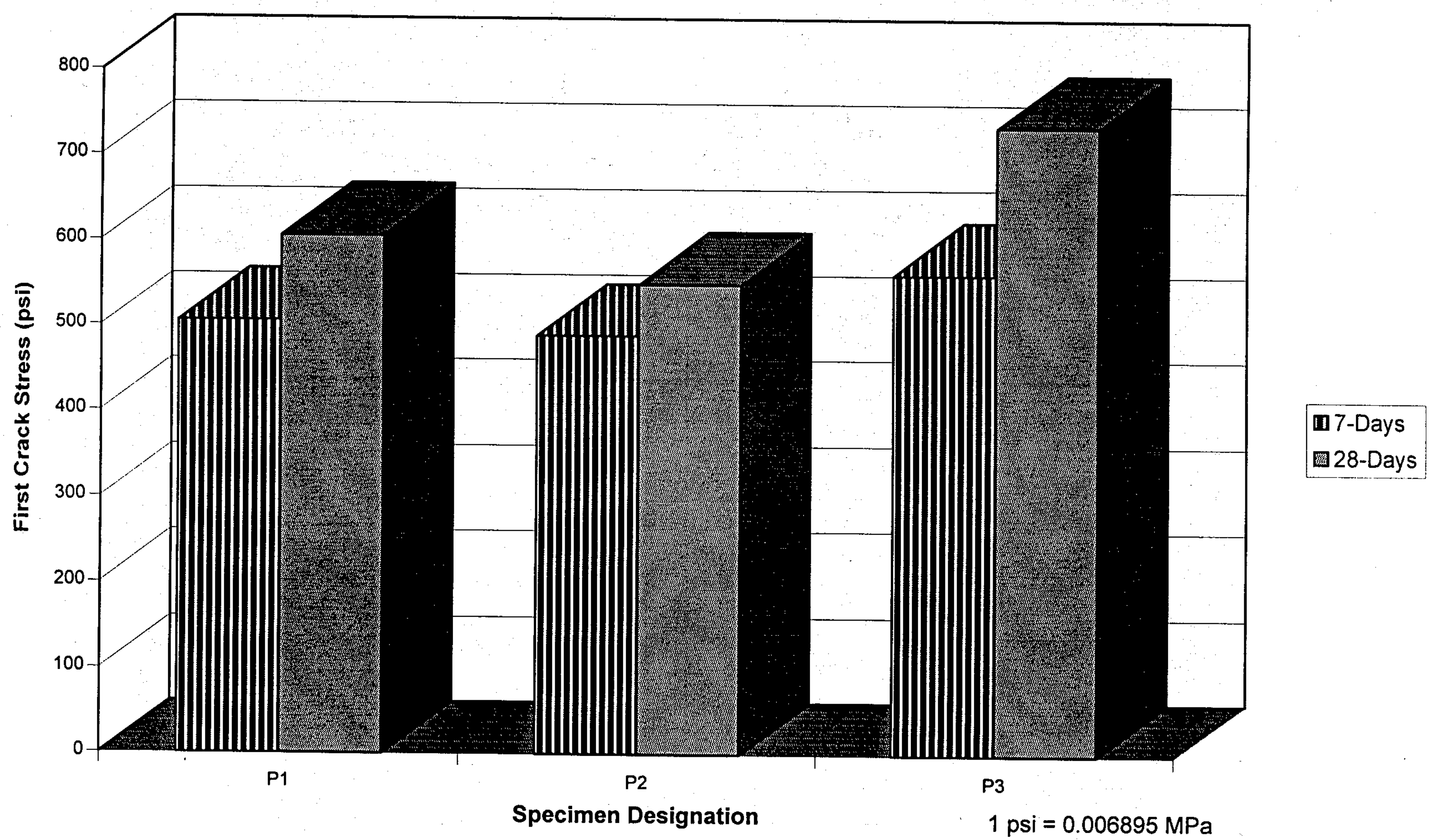
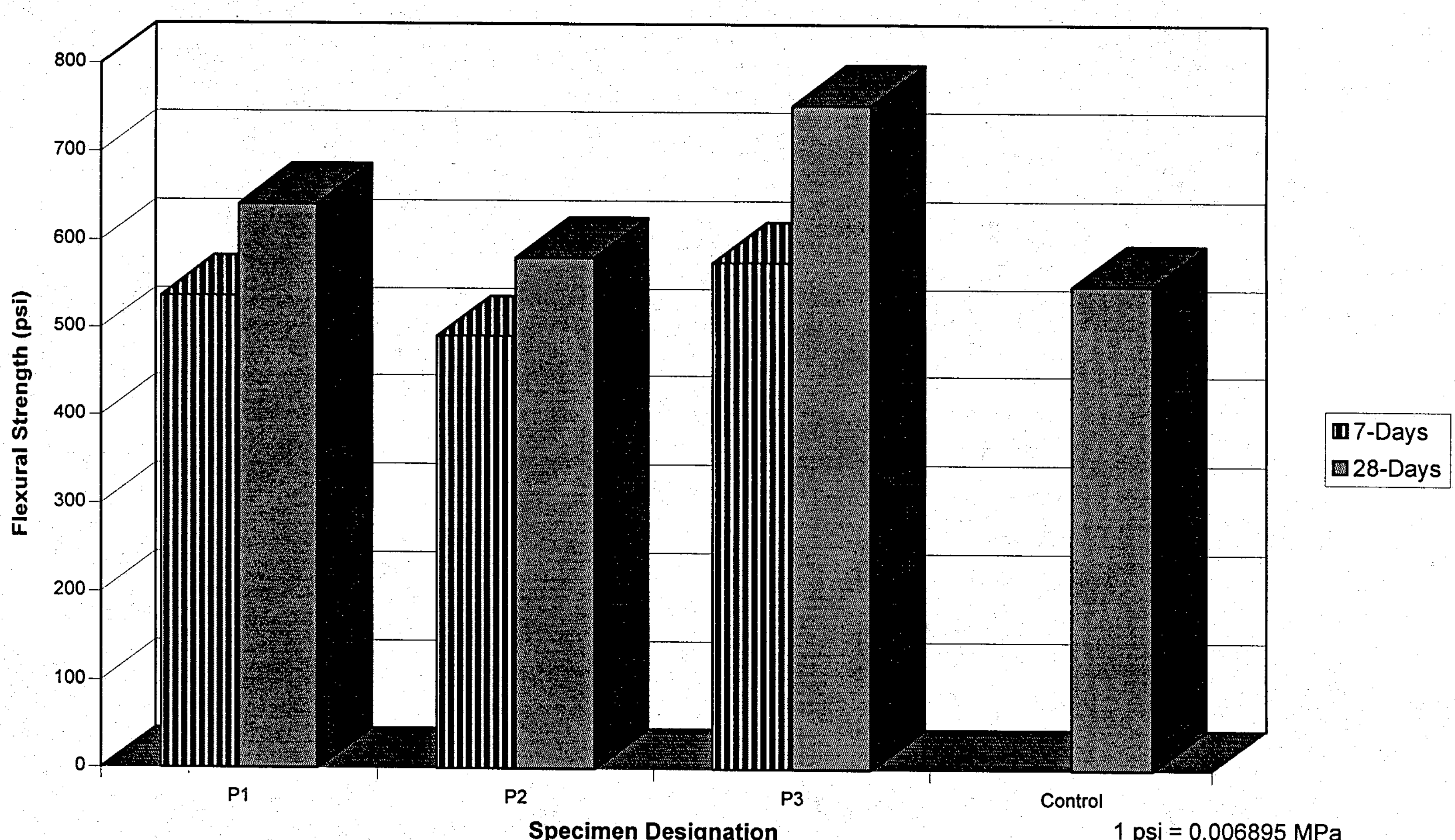


Fig B5: Comparison of Flexural Strength for Different Sections



1 psi = 0.006895 MPa

Fig B6: Comparison of ASTM First Crack Toughness for Different Sections

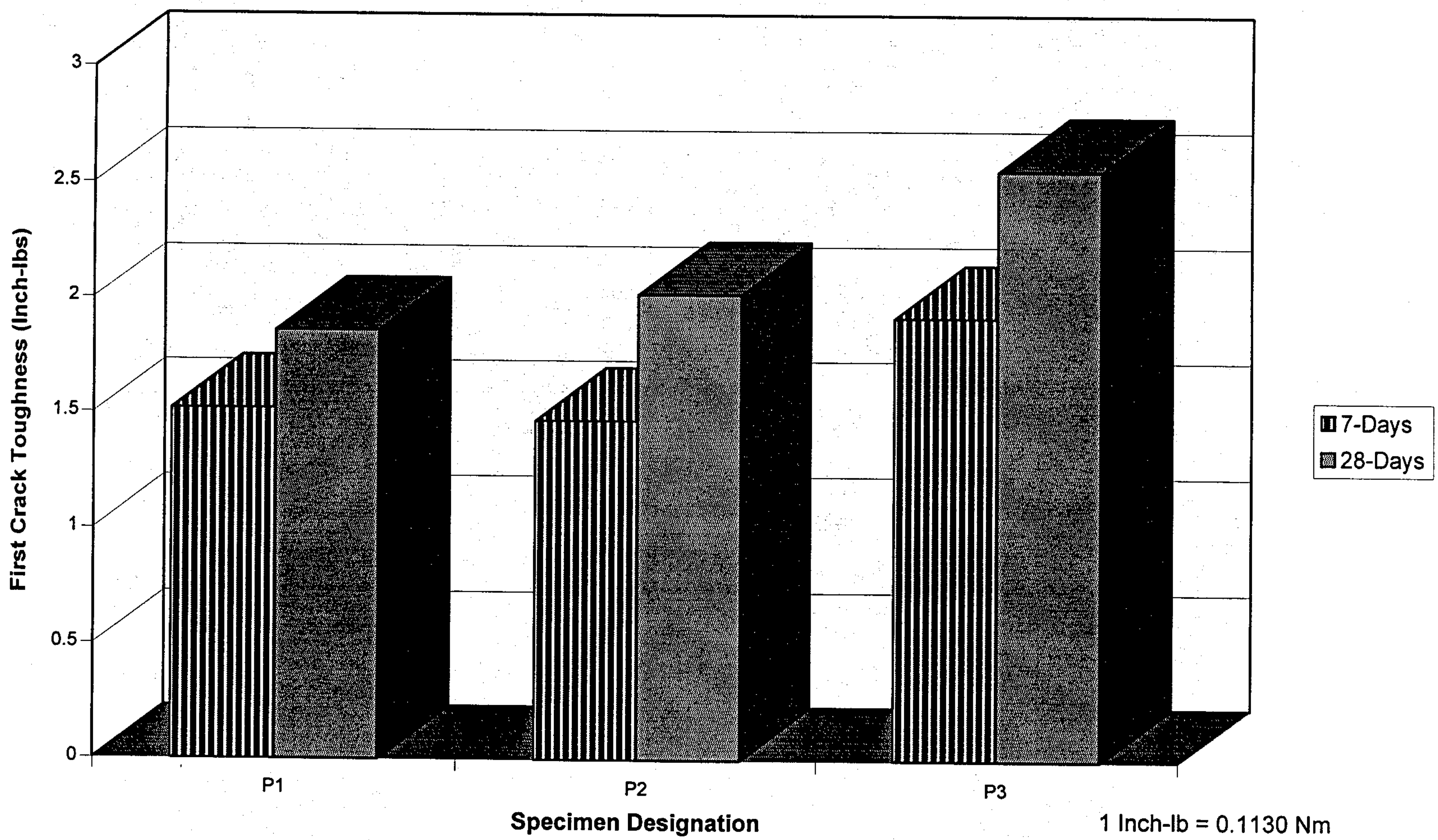


Fig B7: Comparison of 7 Days ASTM Toughness Indices, I5, I10, I20 for Different Sections

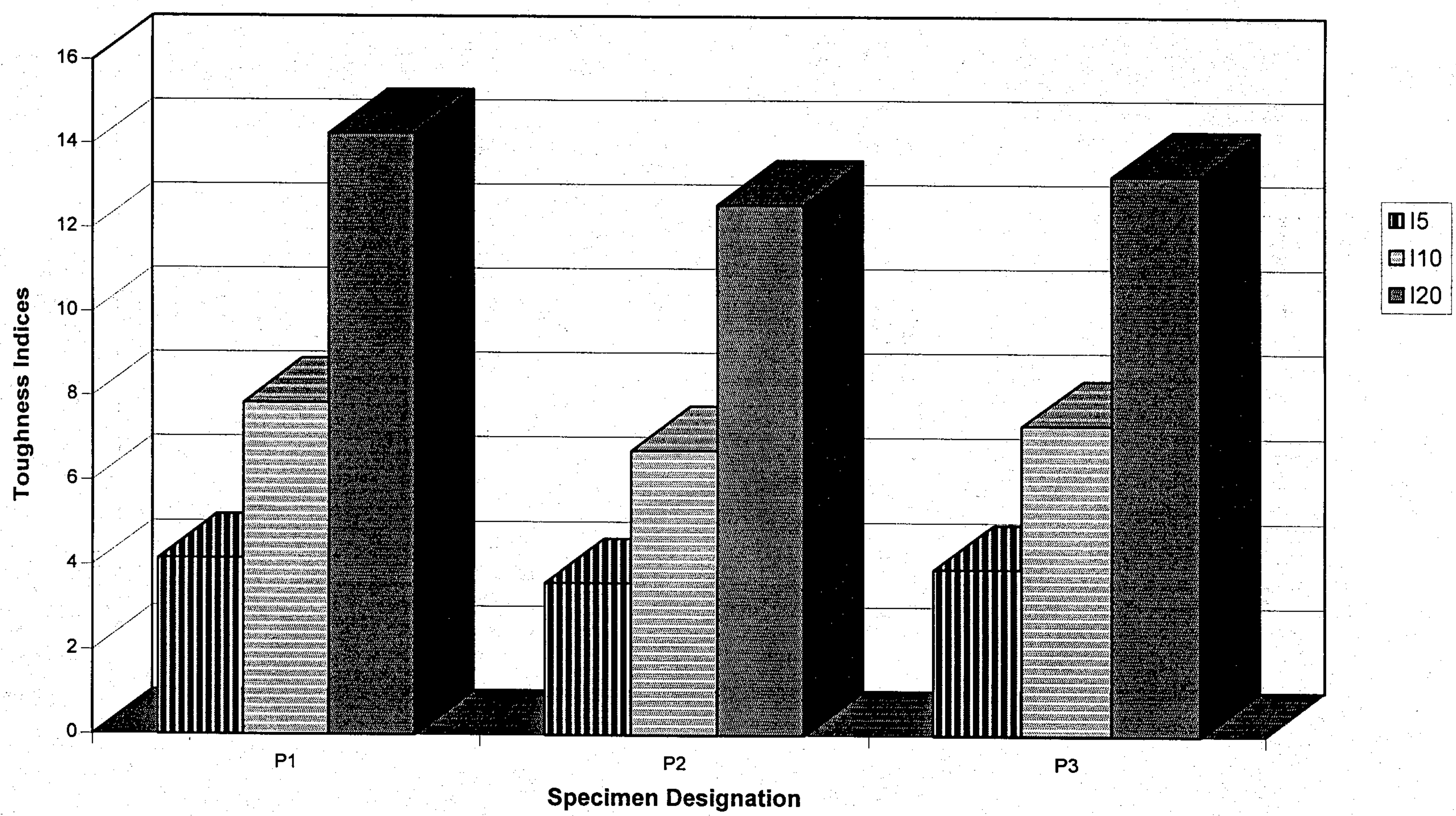


Fig B8: Comparison of 28 Days ASTM Toughness Indices, I5, I10, I20 for Different Sections

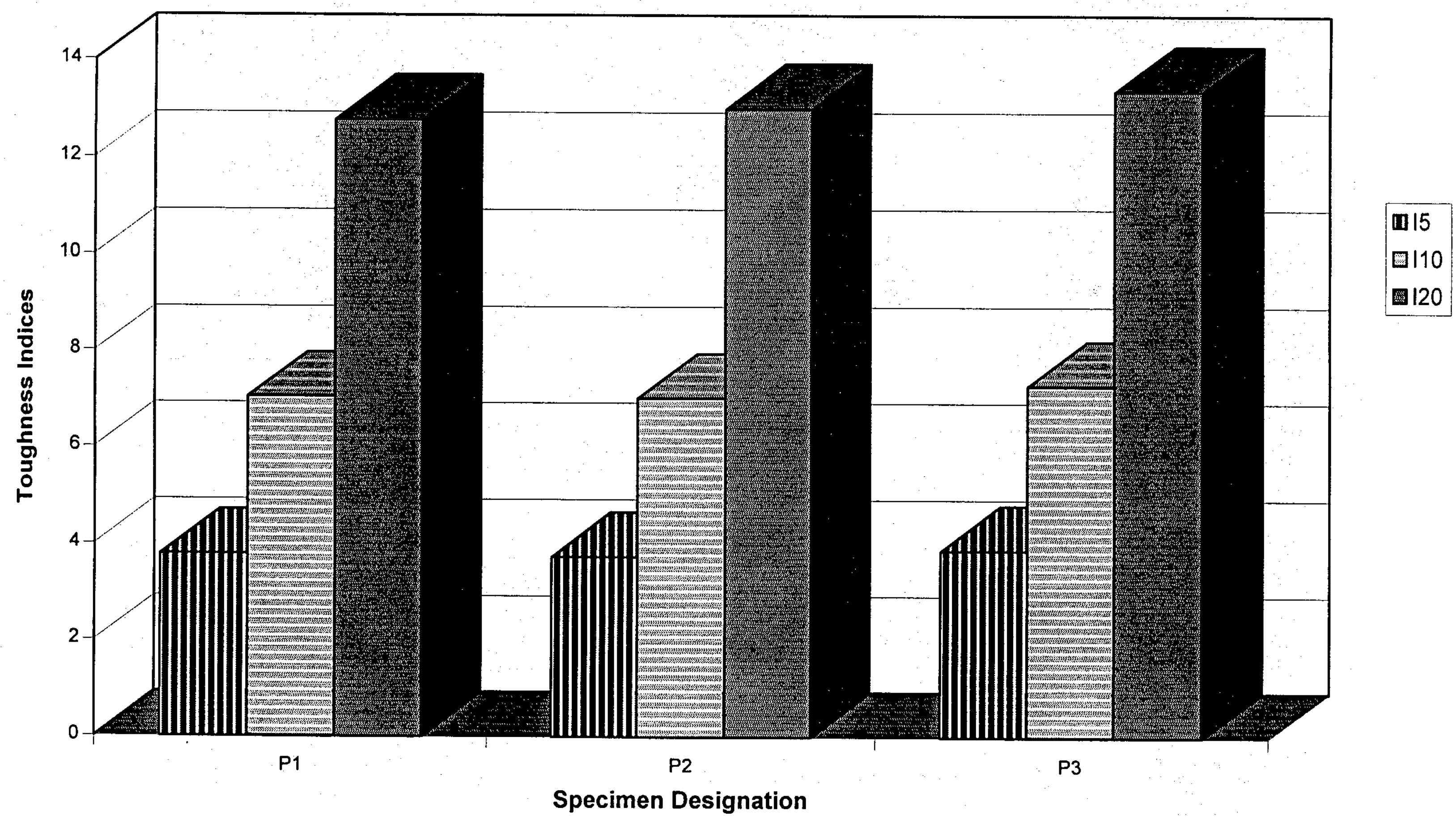


Fig B9: Comparison of 7 and 28 Days Toughness Ratio I10/I5 for Different Sections

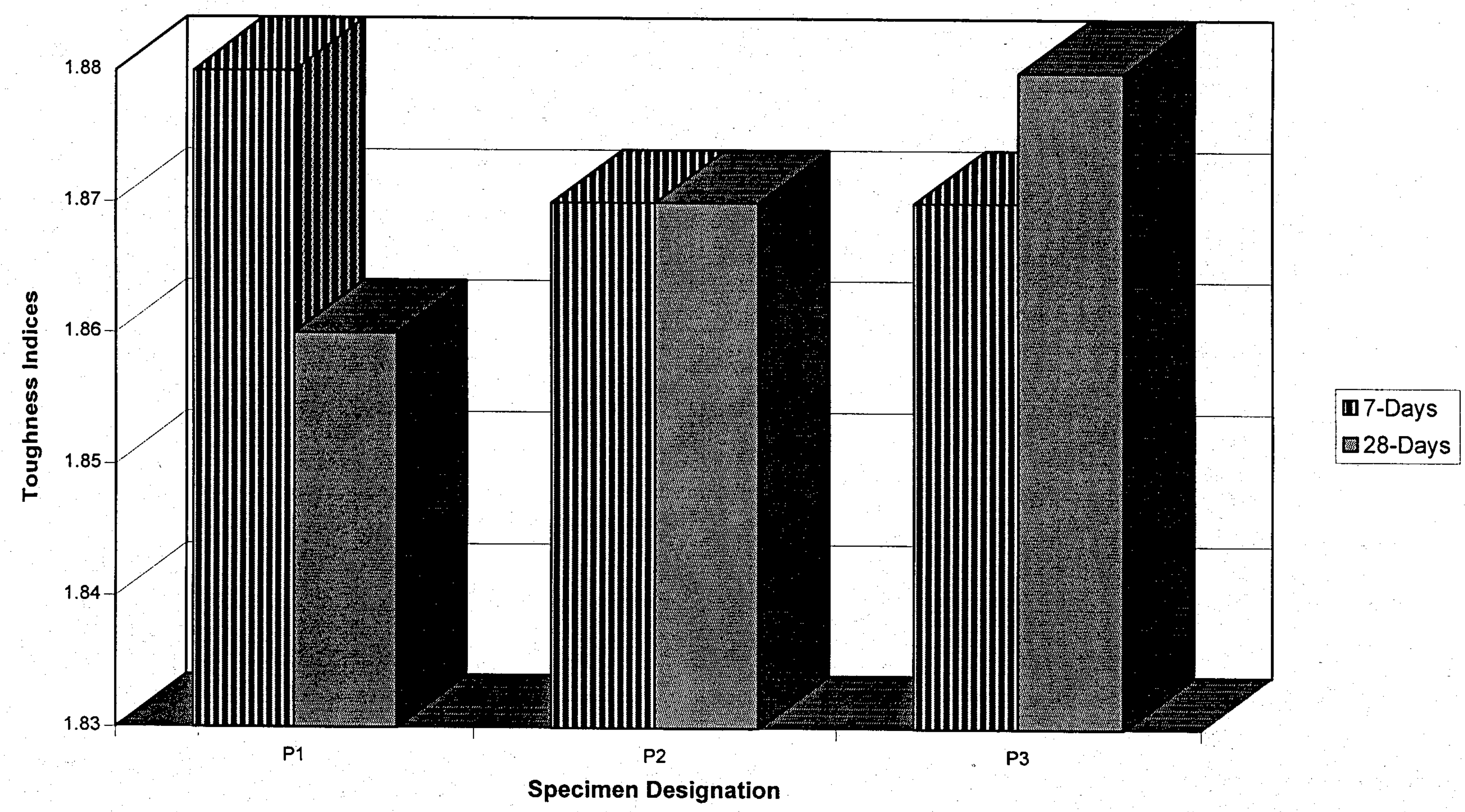


Fig B10: Comparison of 7 and 28 Days Toughness Ratio I20/I10 for Different Sections

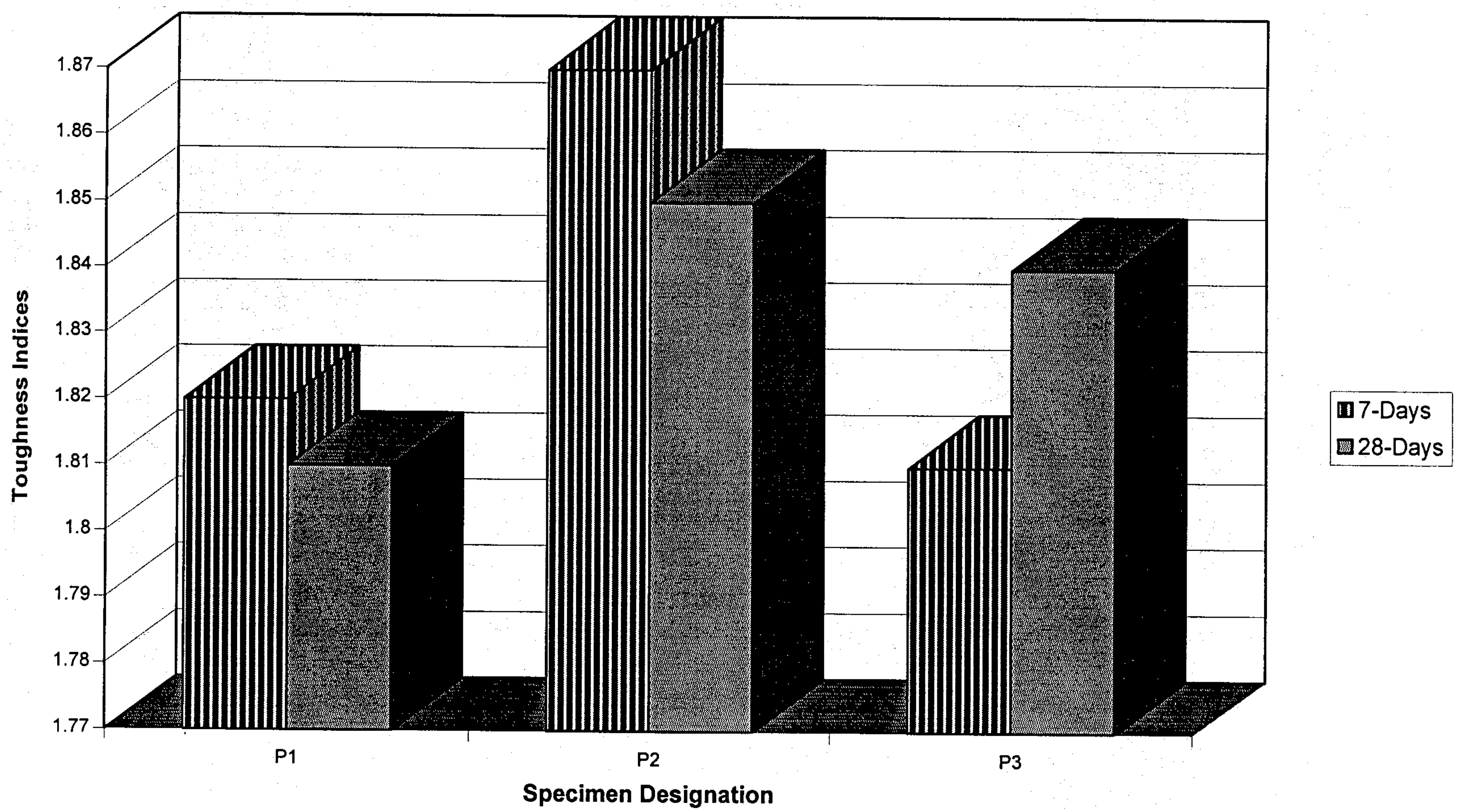


Fig B11: Comparison of Japanese Toughness for Different Sections

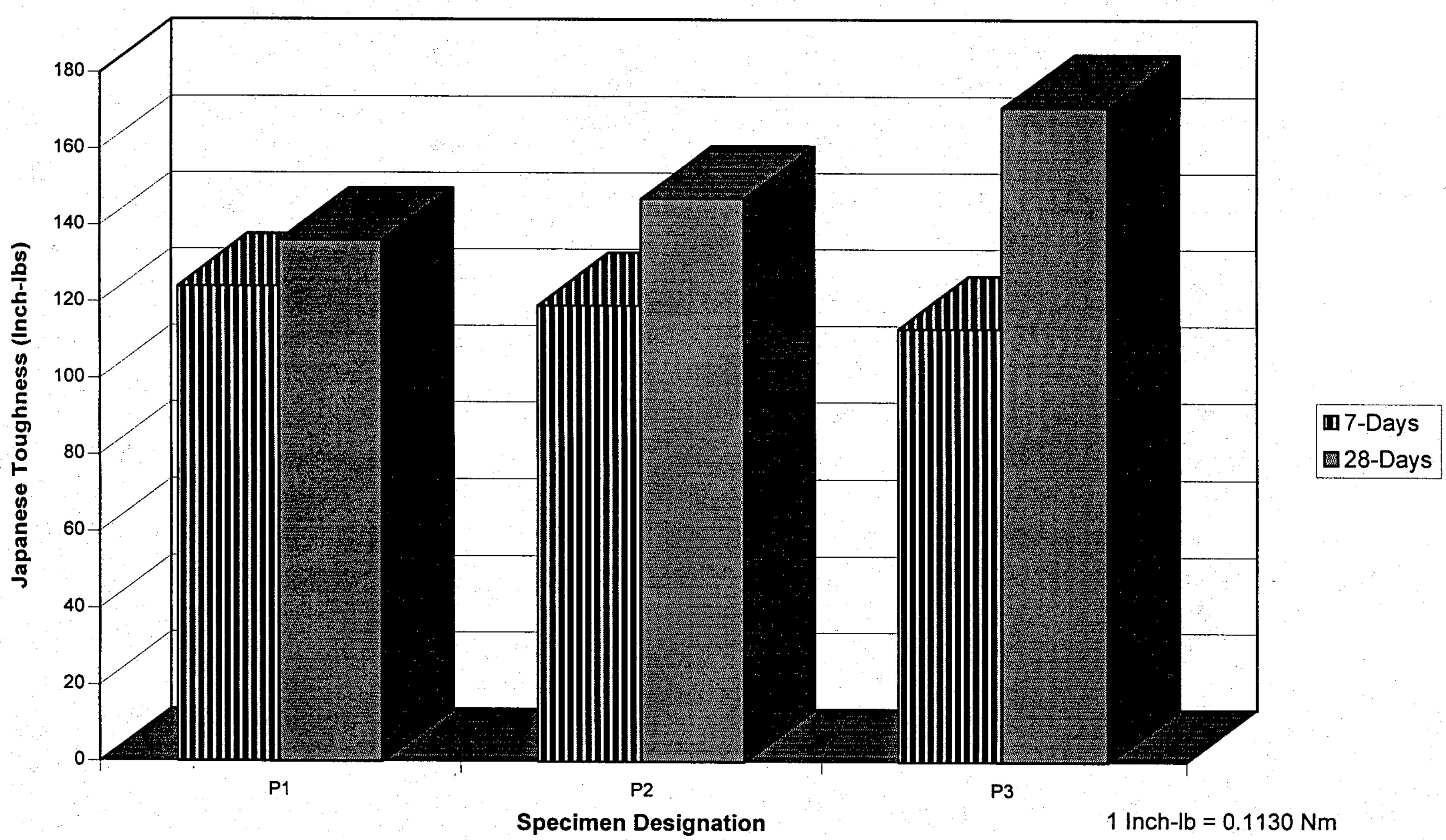
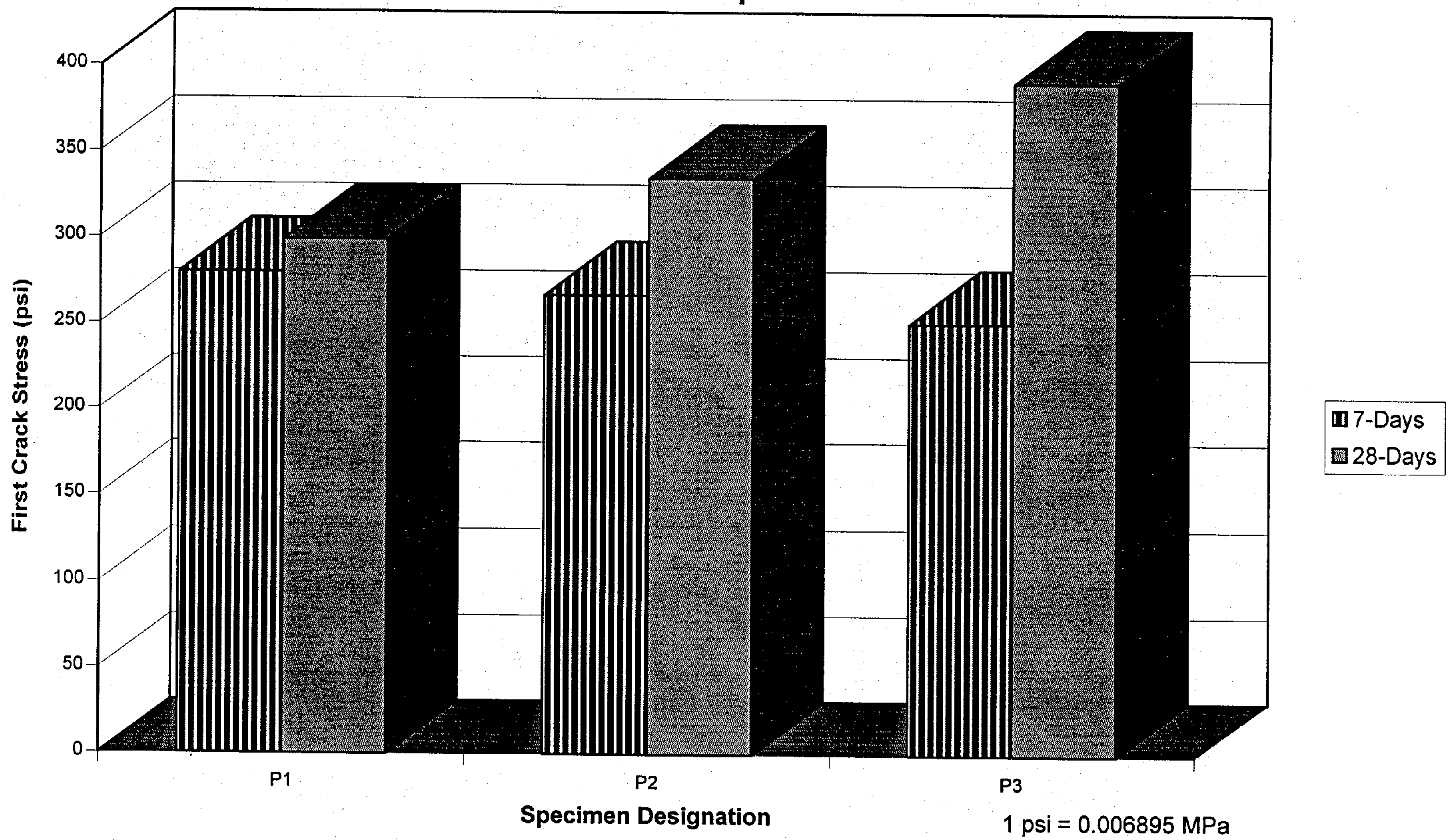


Fig B12: Comparison of Japanese Standard Equivalent Flexural Strength for Different Specimens



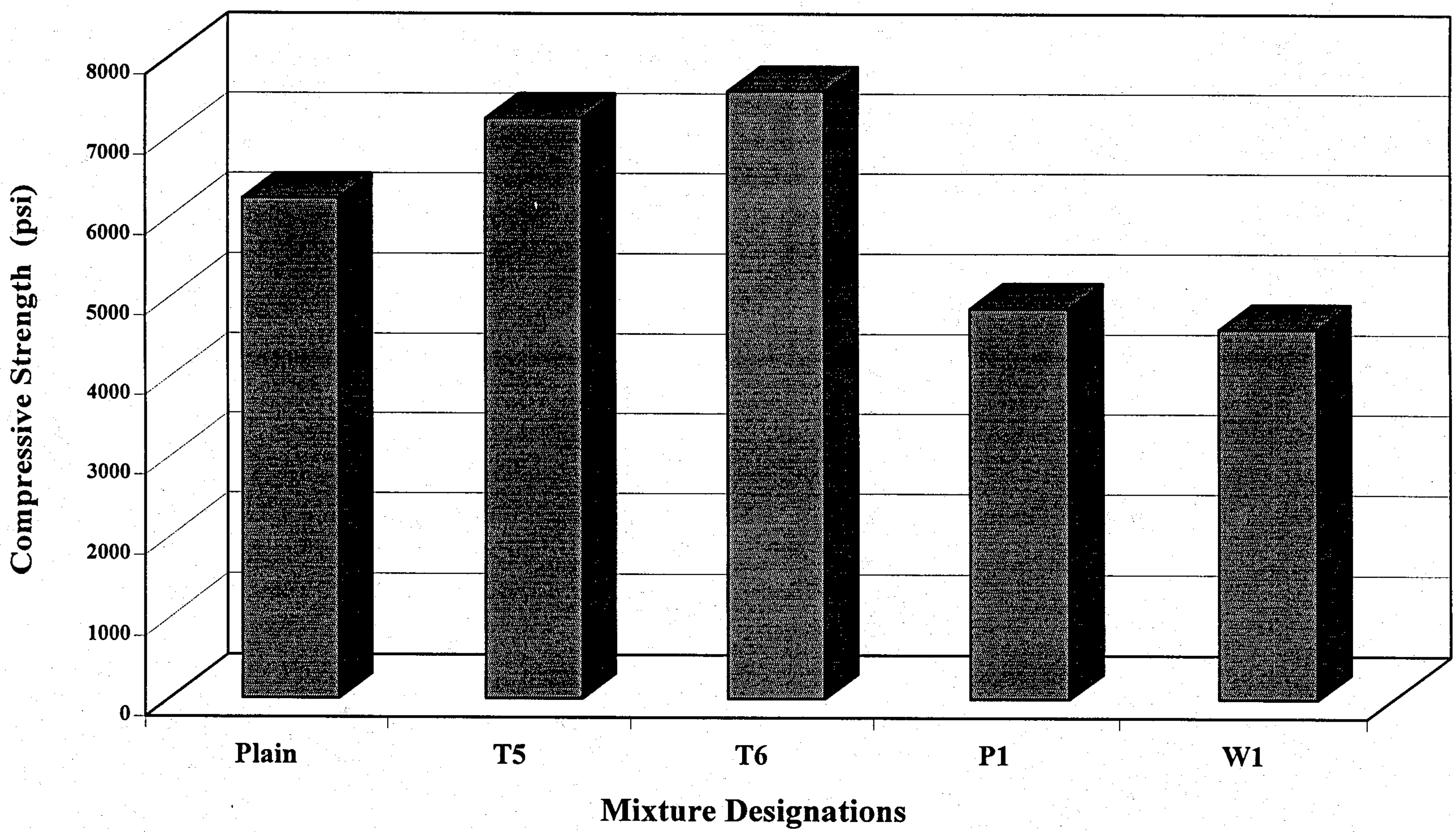


Fig. B13 COMPARISON OF COMPRESSIVE STRENGTH FOR COMBINED MIXTURES

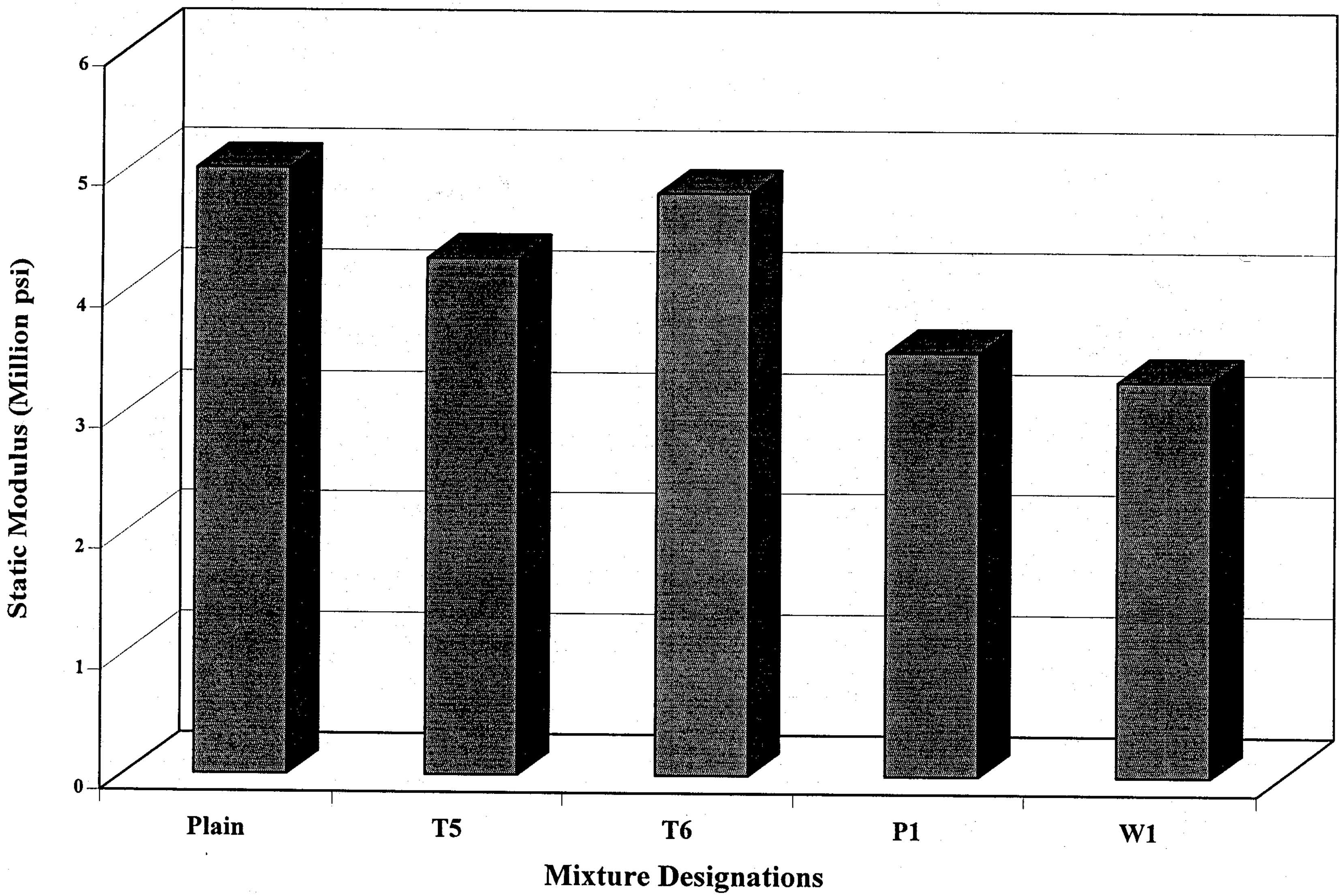


Fig. B14 COMPARISON OF STATIC MODULUS FOR COMBINED MIXTURES

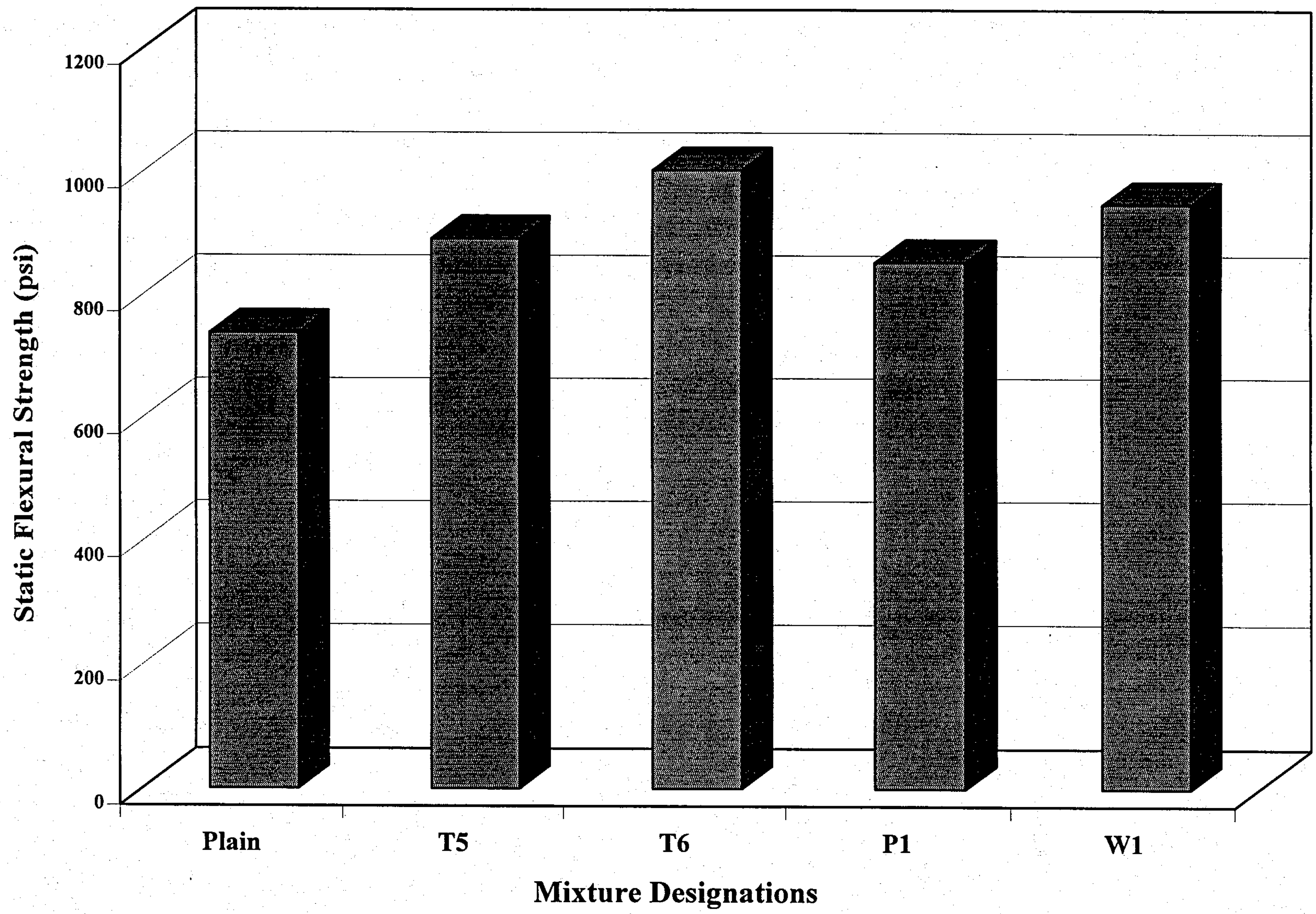


Fig. B15 COMPARISON OF STATIC FLEXURAL STRENGTH FOR COMBINED MIXTURES

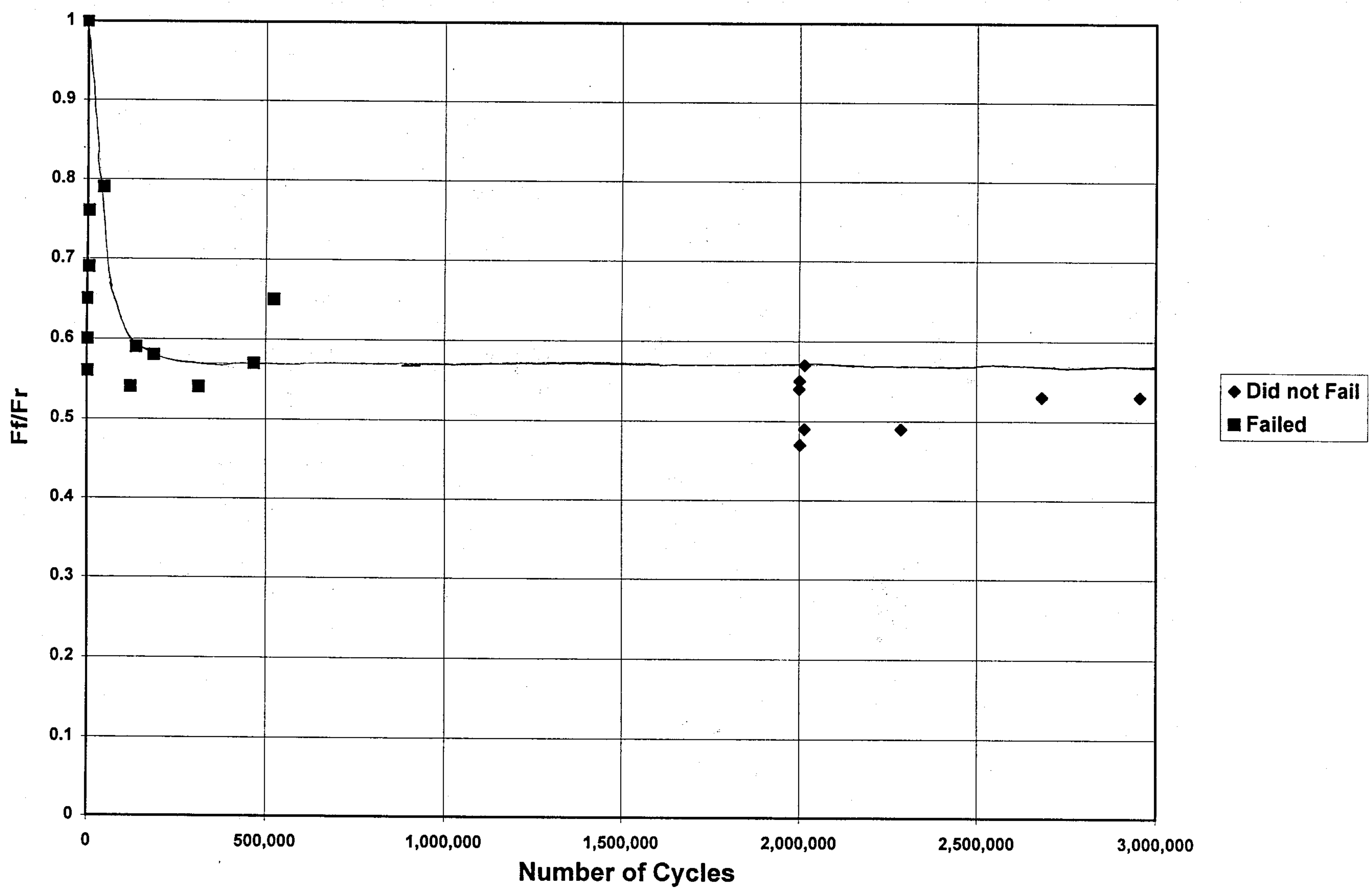


Fig. B16 NUMBER OF CYCLES Vs Ff/Fr FOR MIXTURE DOT-T5

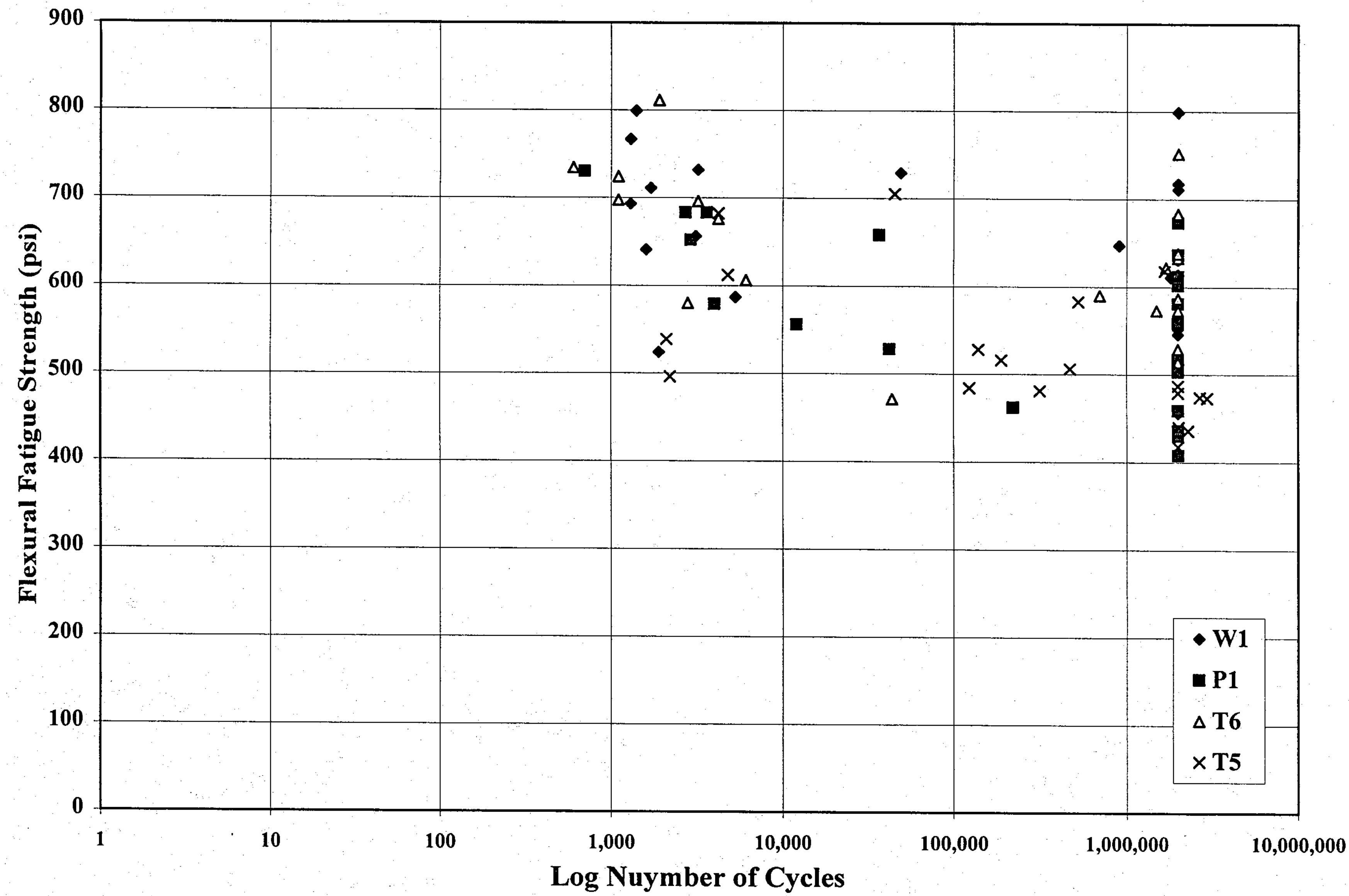


Fig.B17 Ff Vs LOG NUMBER OF CYCLES FOR ALL MIXTURES

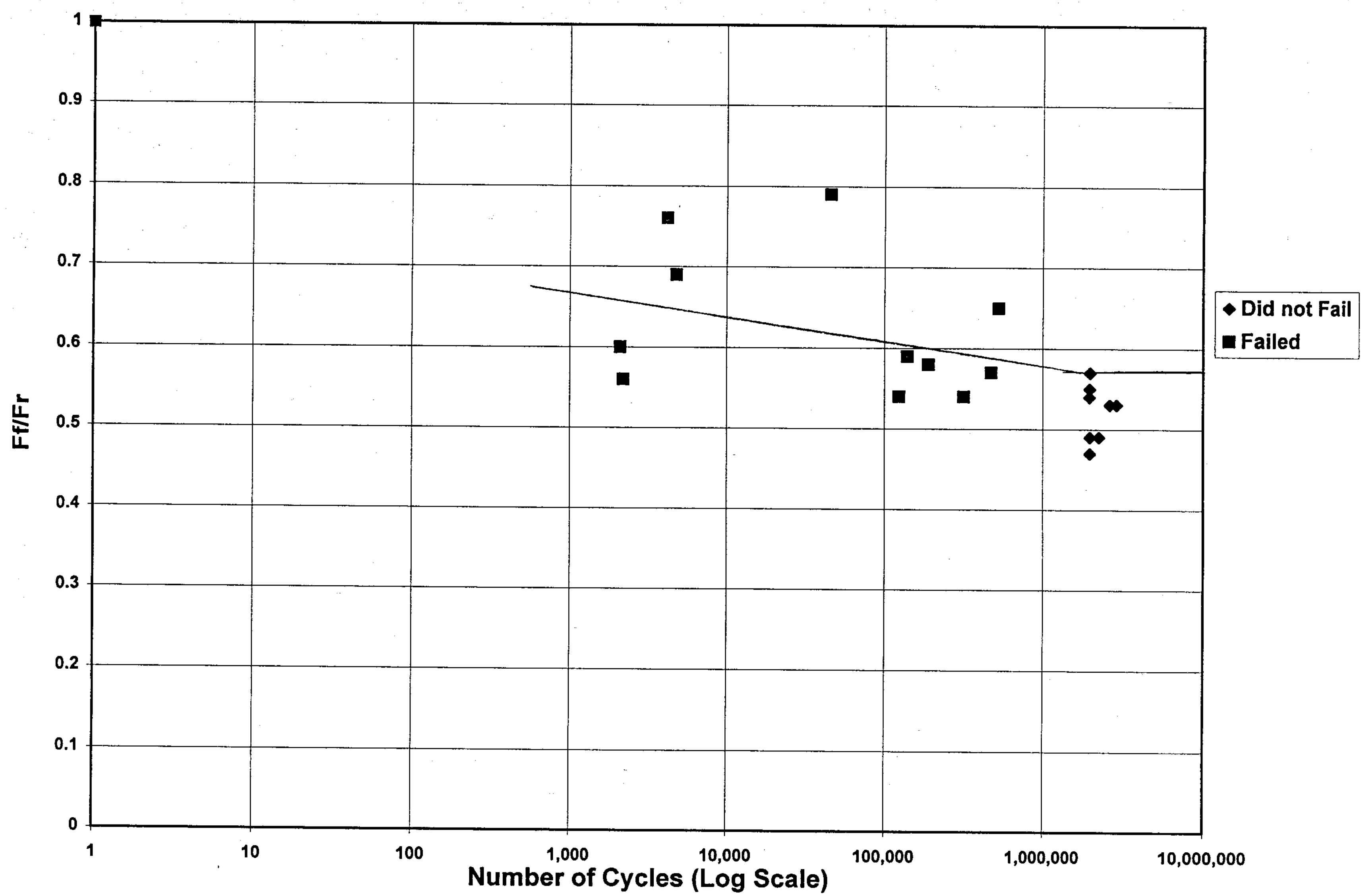


Fig.B18 NUMBER OF CYCLES Vs Ff/Fr FOR MIXTURE DOT-T5

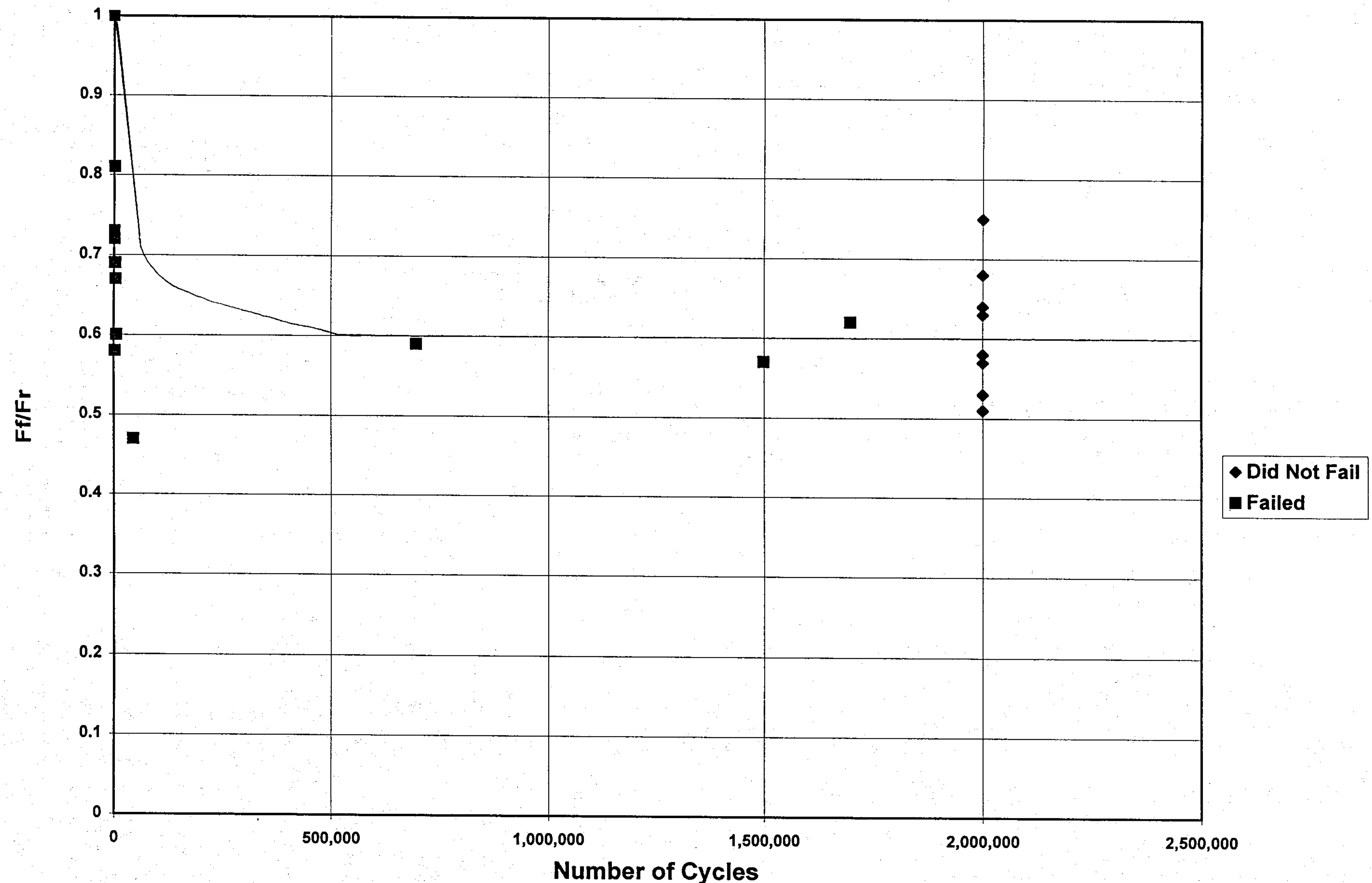


Fig.B19 NUMBER OF CYCLES Vs Ff/Fr FOR MIXTURE DOT-T6

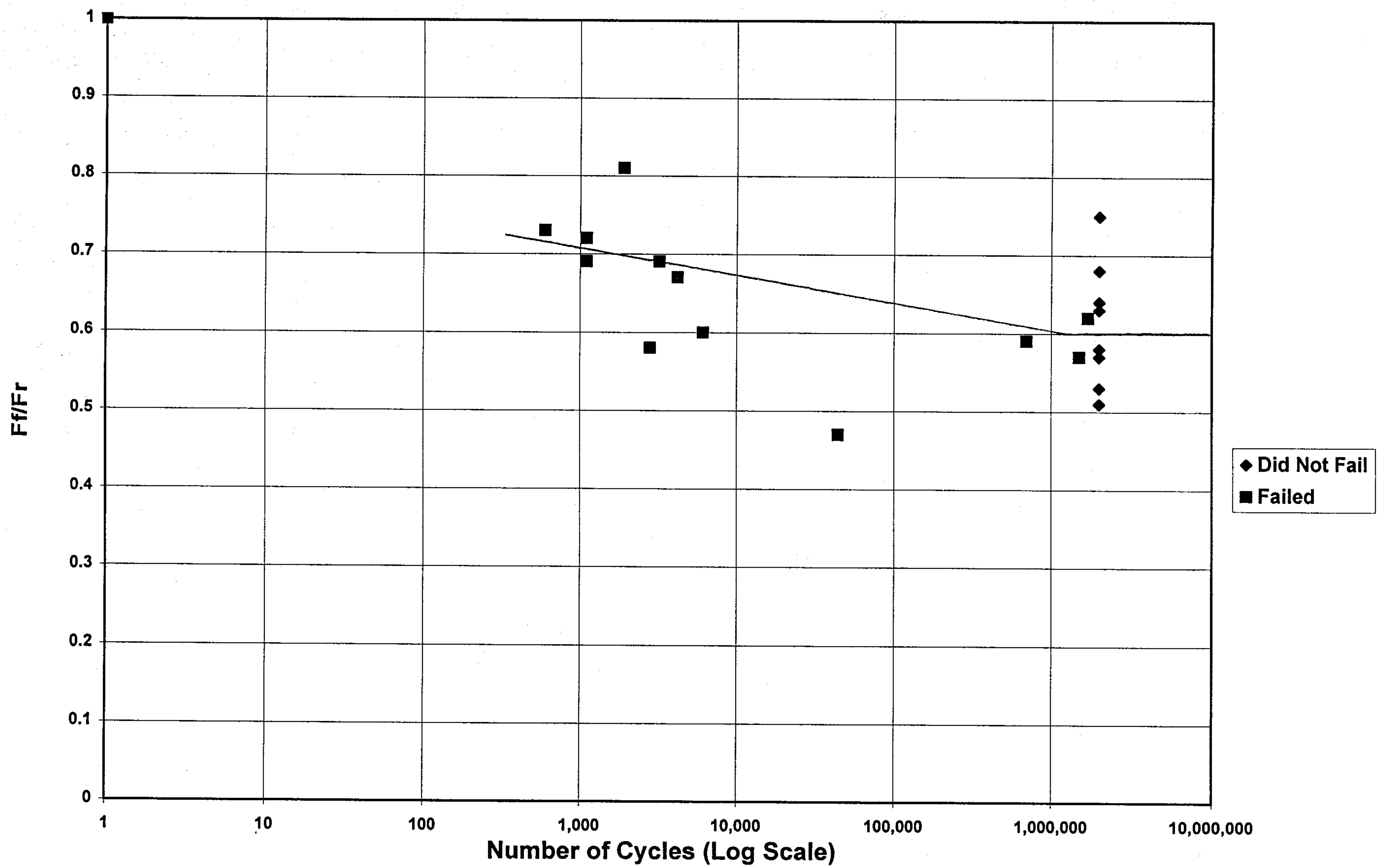


Fig.B20 NUMBER OF CYCLES Vs Ff/Fr FOR MIXTURE DOT-T6

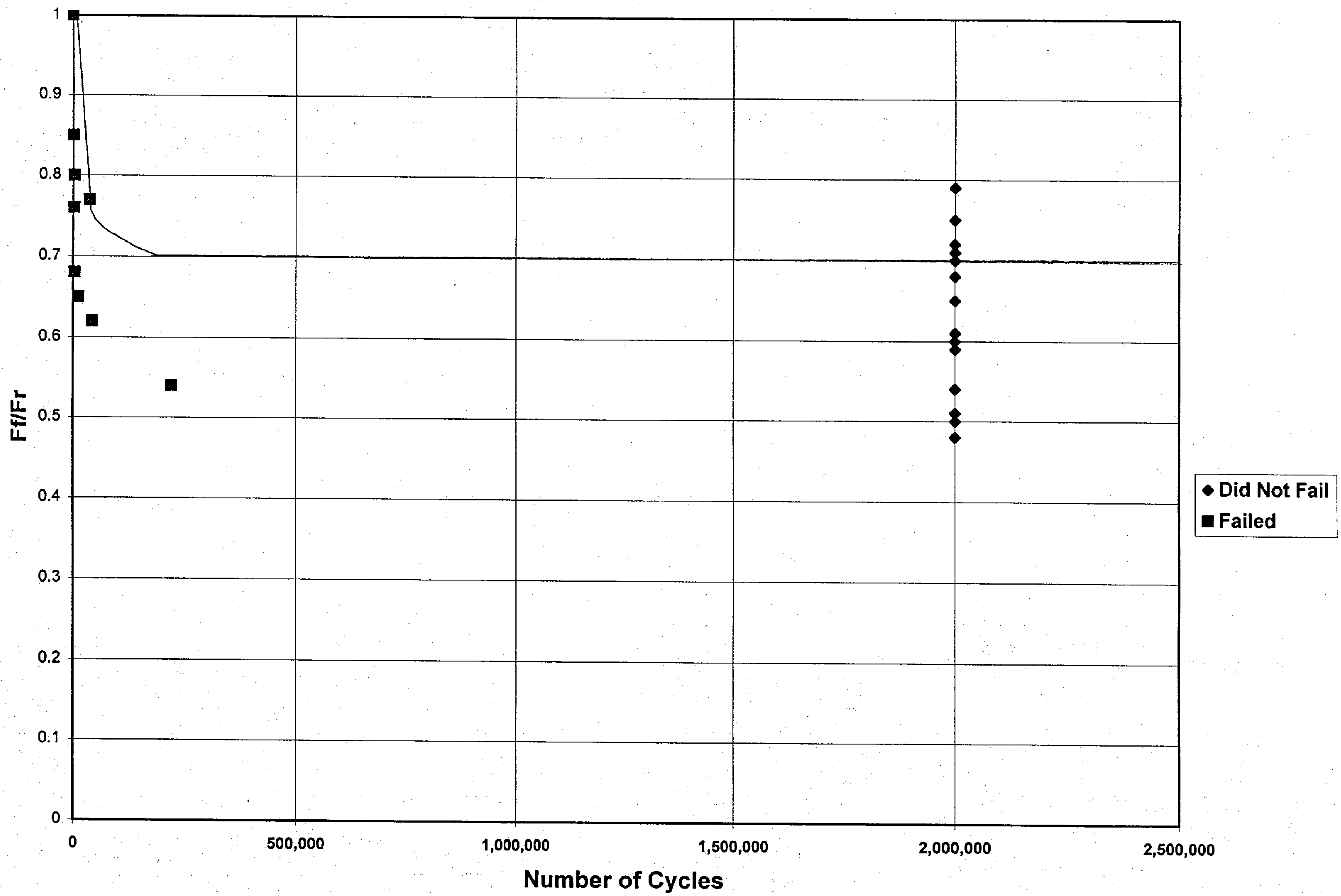


Fig.B21 NUMBER OF CYCLES Vs Ff/Fr FOR MIXTURE DOT-P1

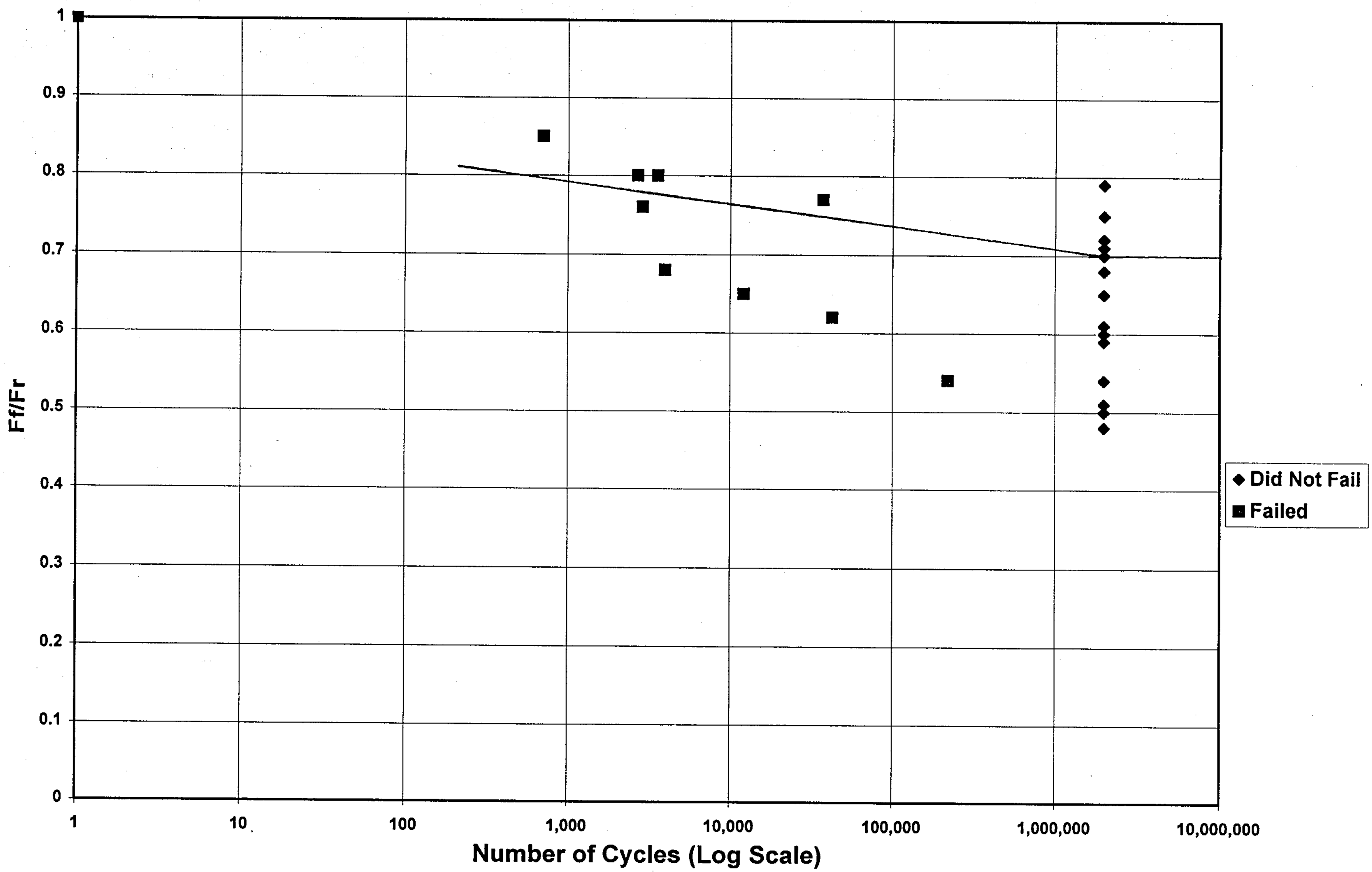


Fig. B22 NUMBER OF CYCLES Vs Ff/Fr FOR MIXTURE DOT-P1

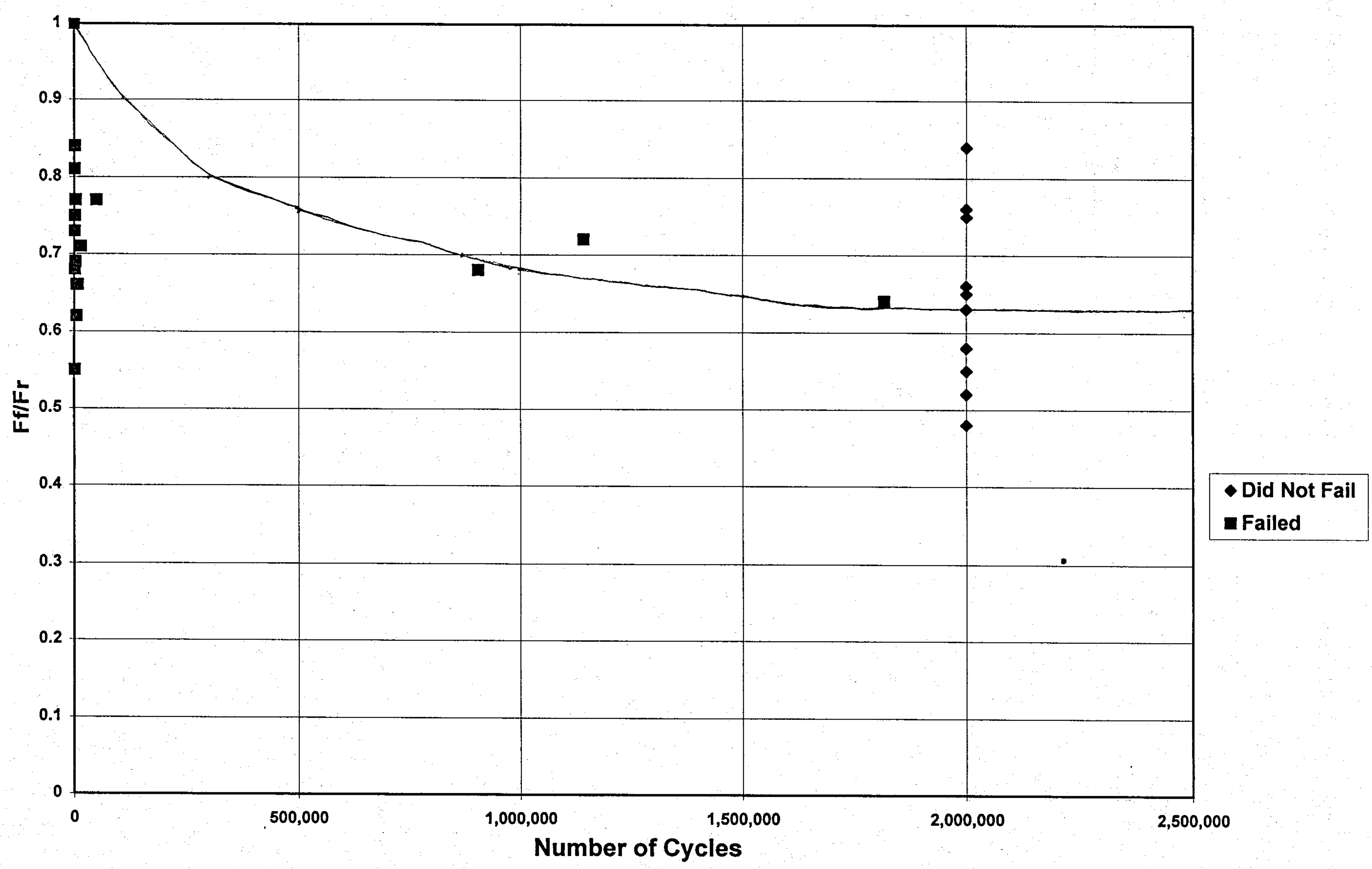


Fig. B23 NUMBER OF CYCLES Vs Ff/Fr FOR MIXTURE DOT-W1

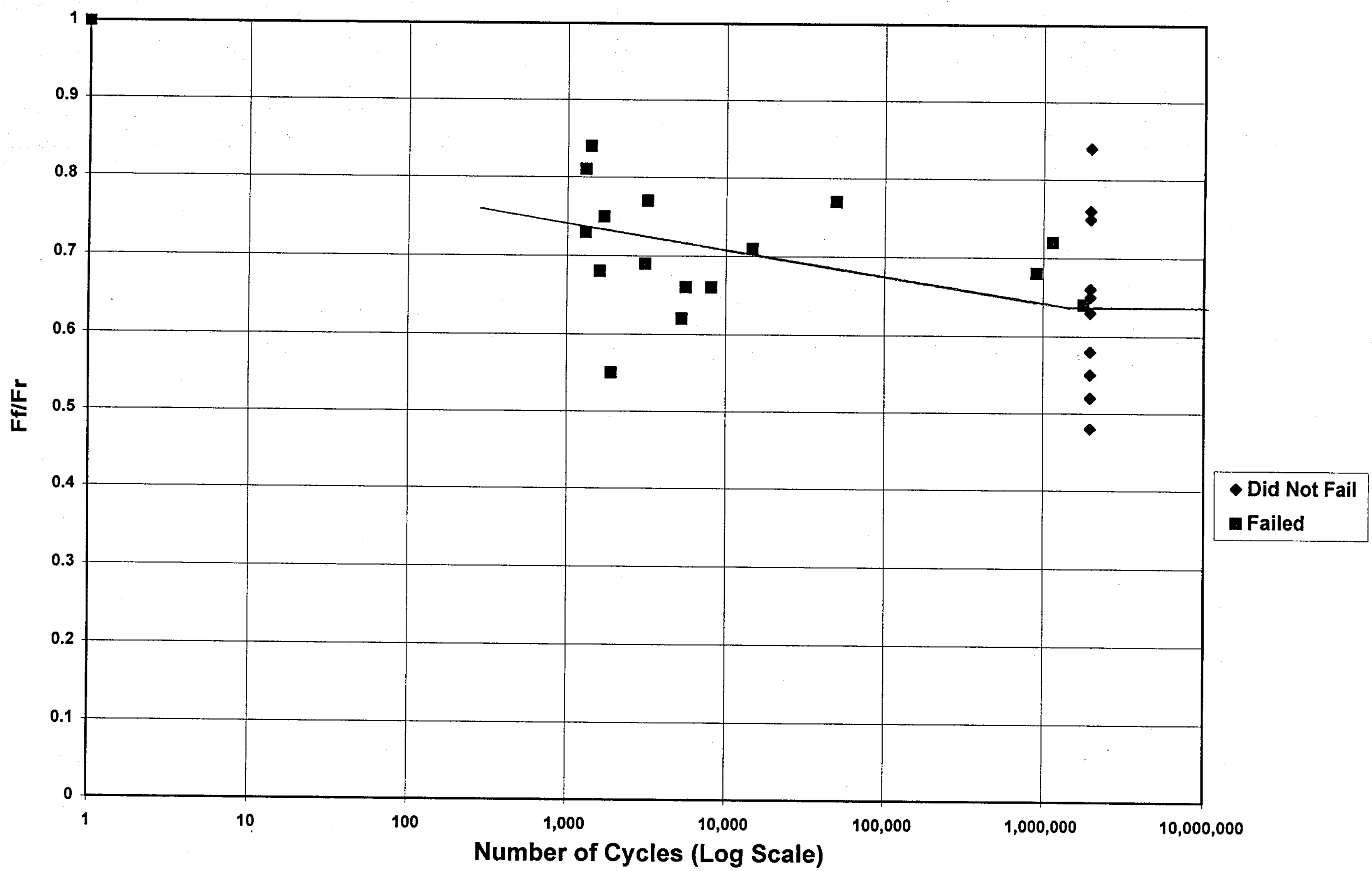


Fig.B24 NUMBER OF CYCLES Vs Ff/Fr FOR MIXTURE DOT-W1

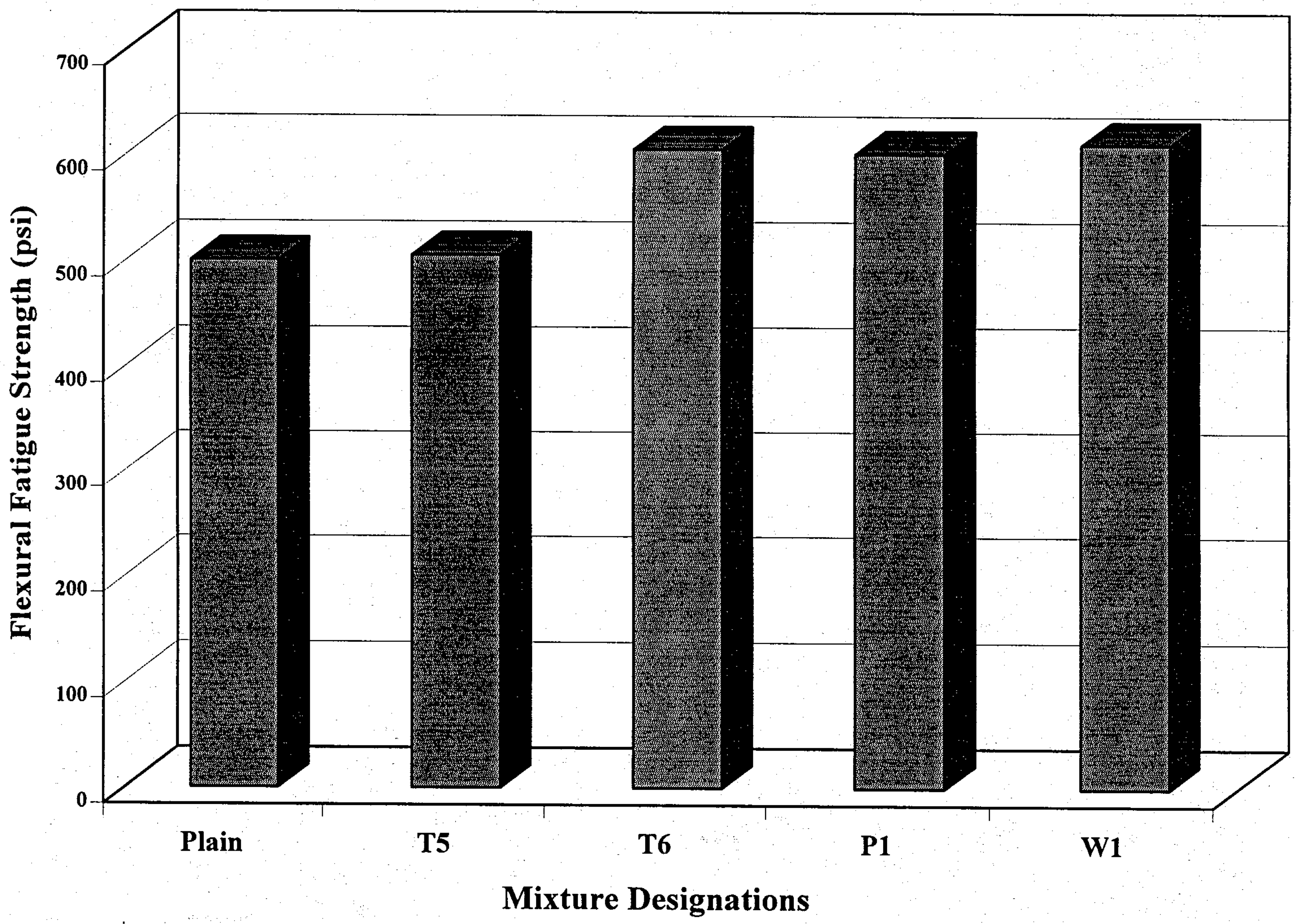


Fig.B25 COMPARISON OF FLEXURAL FATIGUE STRENGTH

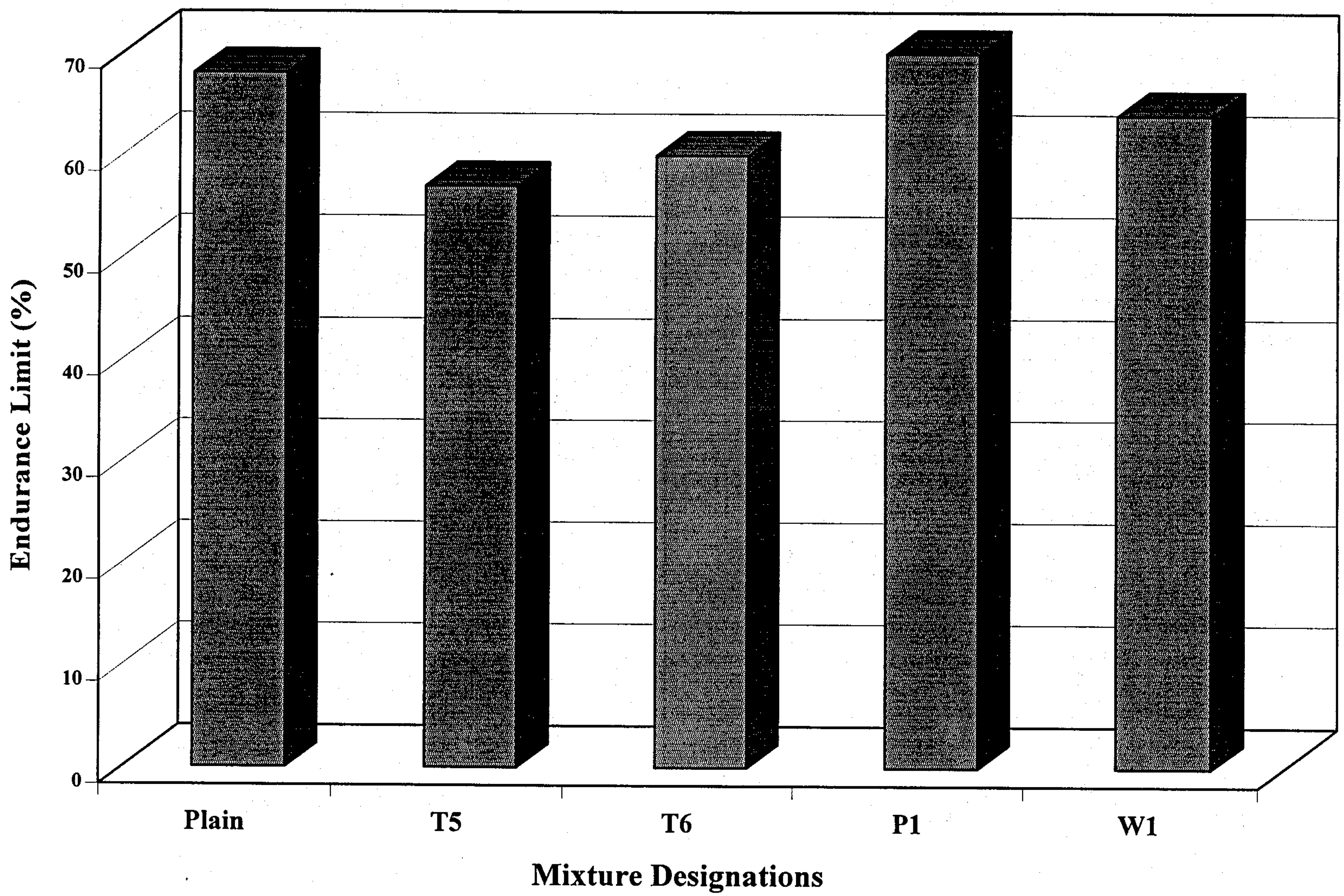


Fig. B26 COMPARISON OF E. L. BASED ON ITSELF

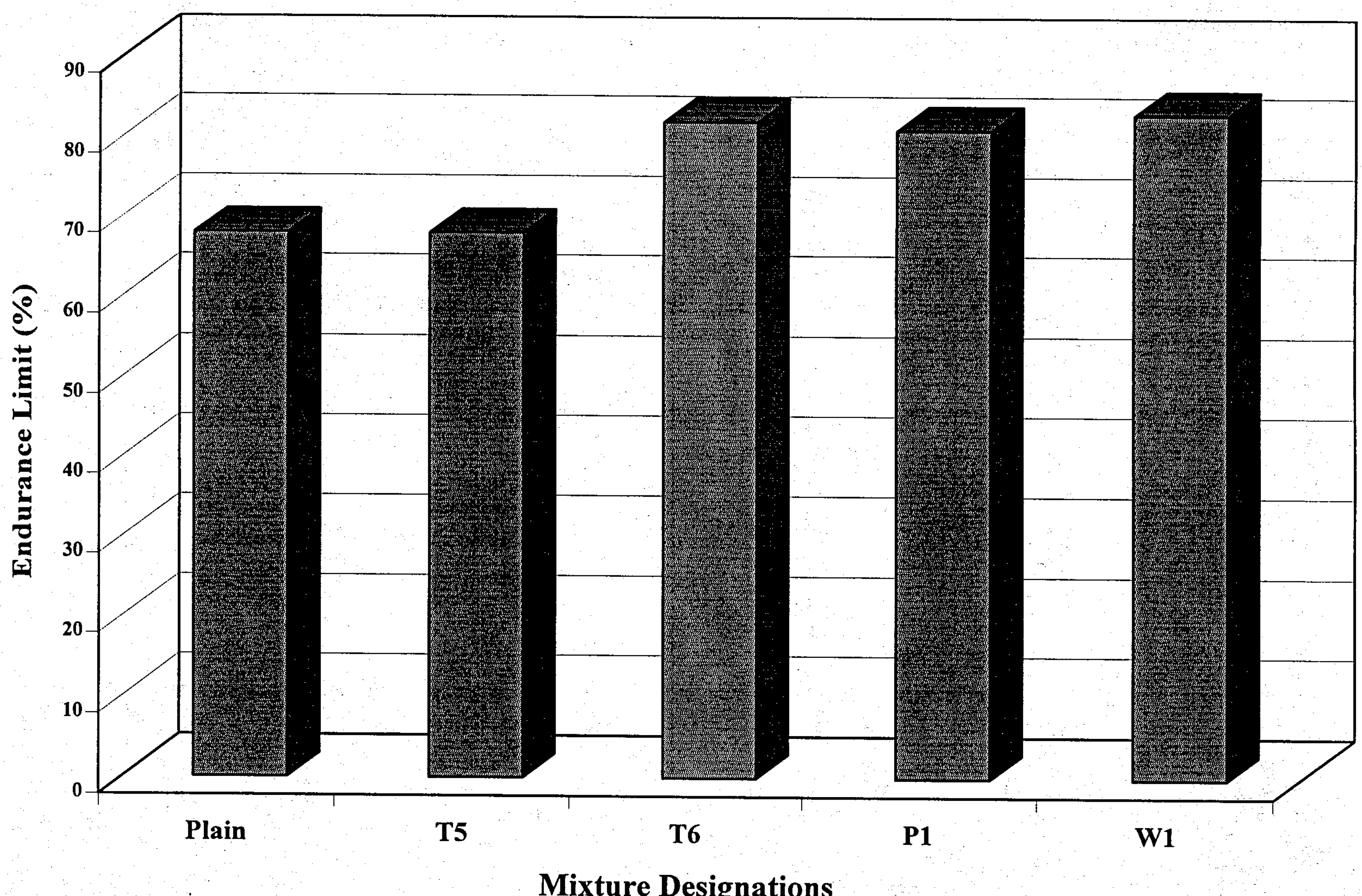


Fig. B27 COMPARISON OF E. L. BASED ON PLAIN CONCRETE

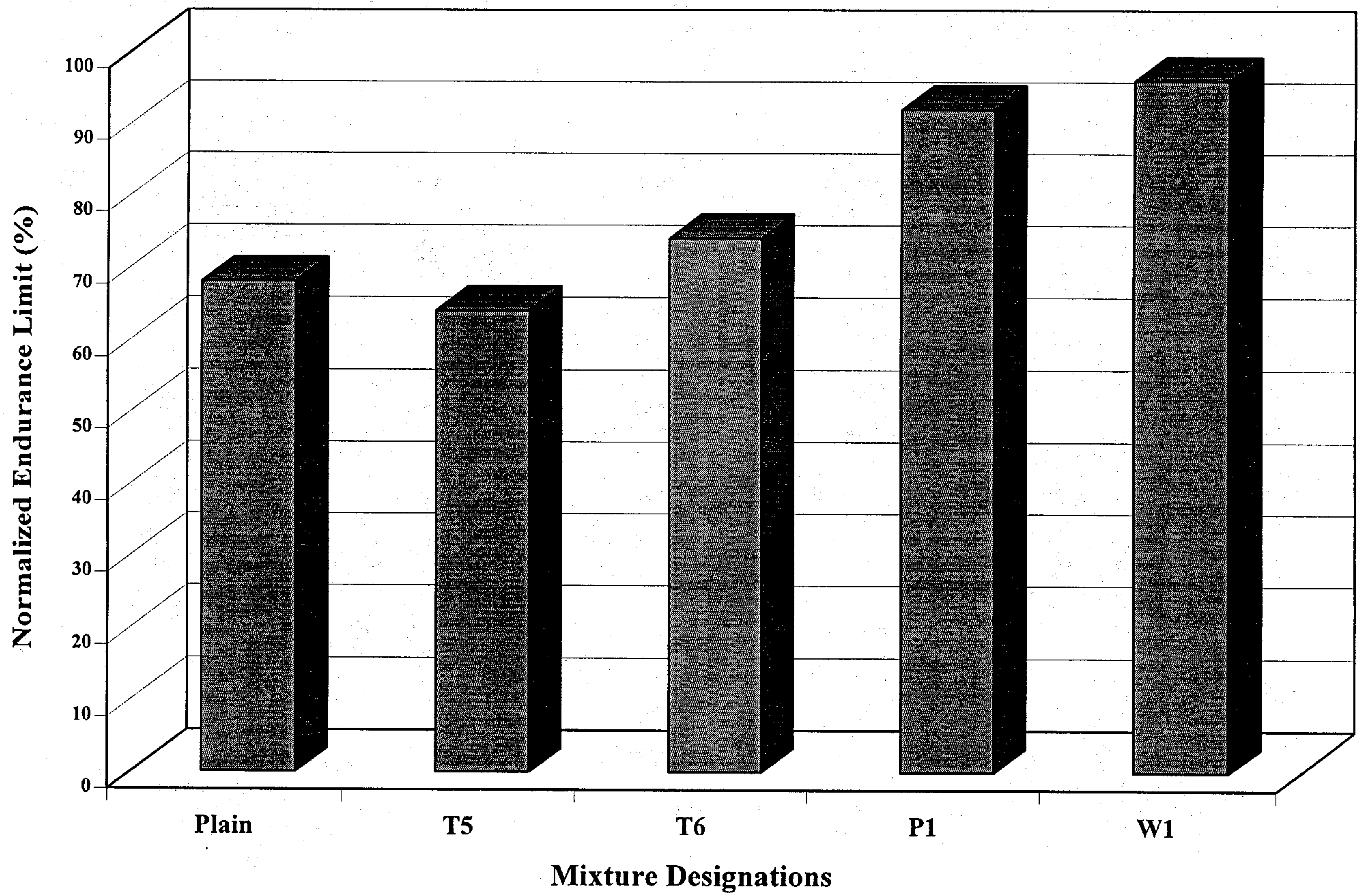


Fig.B28 COMPARISON OF NORMALIZED E. L. BASED ON PLAIN CONCRETE

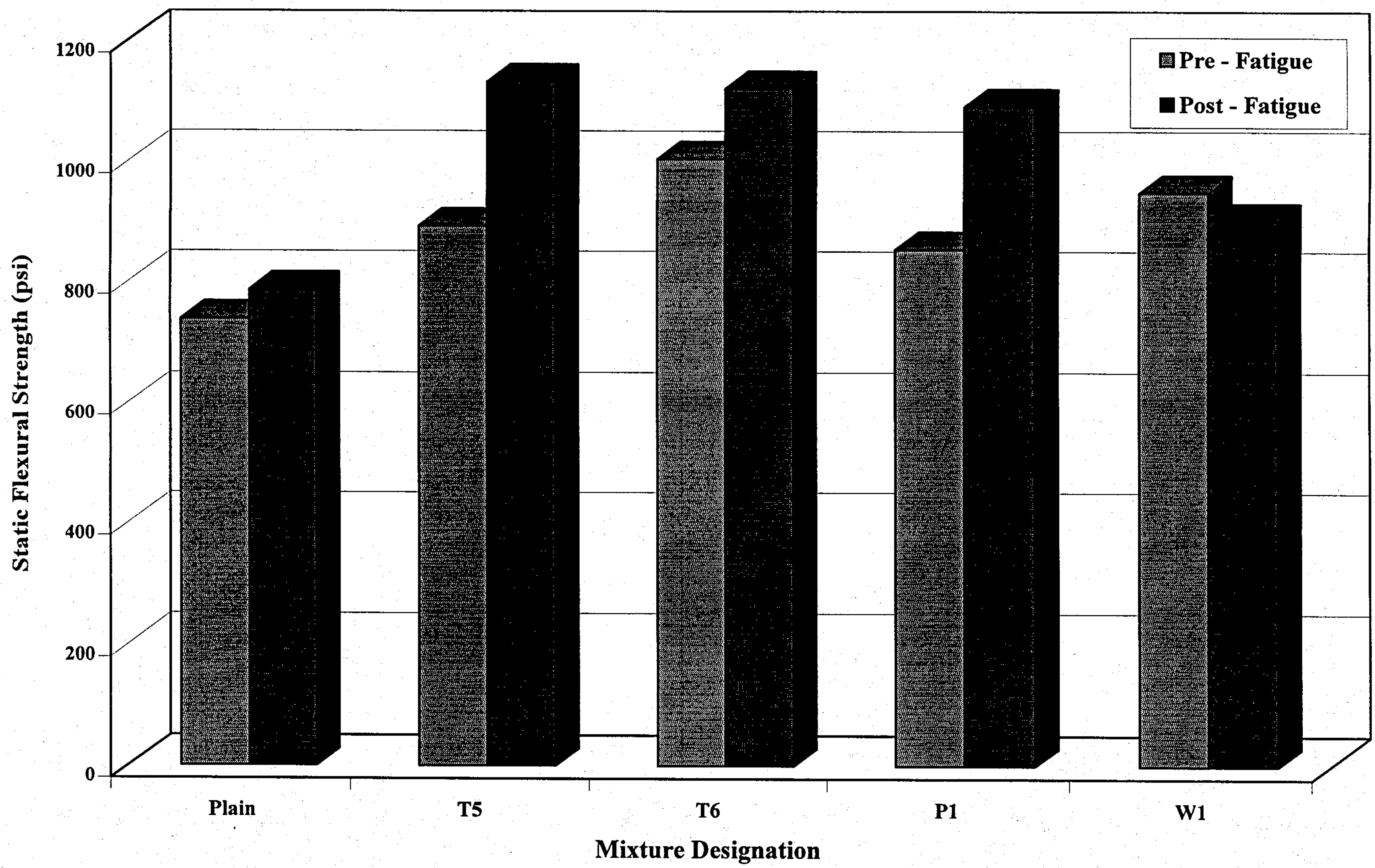
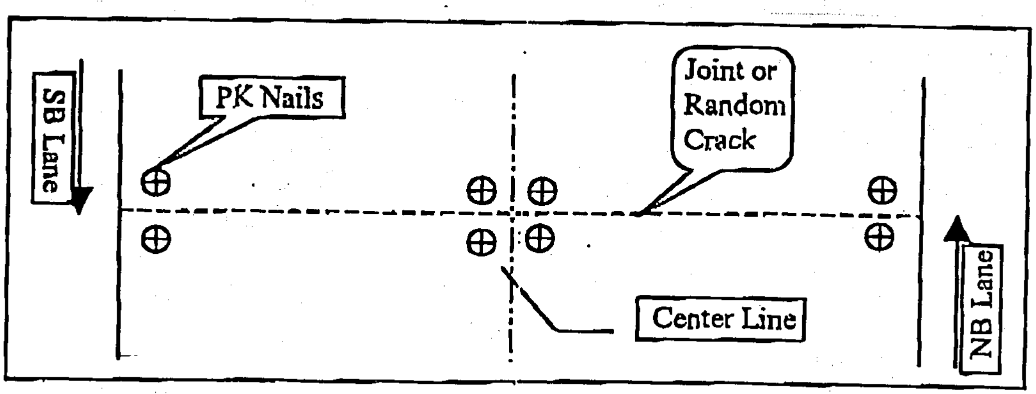


Fig.B29 COMPARISON OF STATIC FLEXURAL STRENGTH BEFORE AND AFTER FATIGUE TESTING

APPENDIX C

Details of the Inspections (Crack Measurement and P.K. Nail Recordings)

The locations of PK nails are shown below



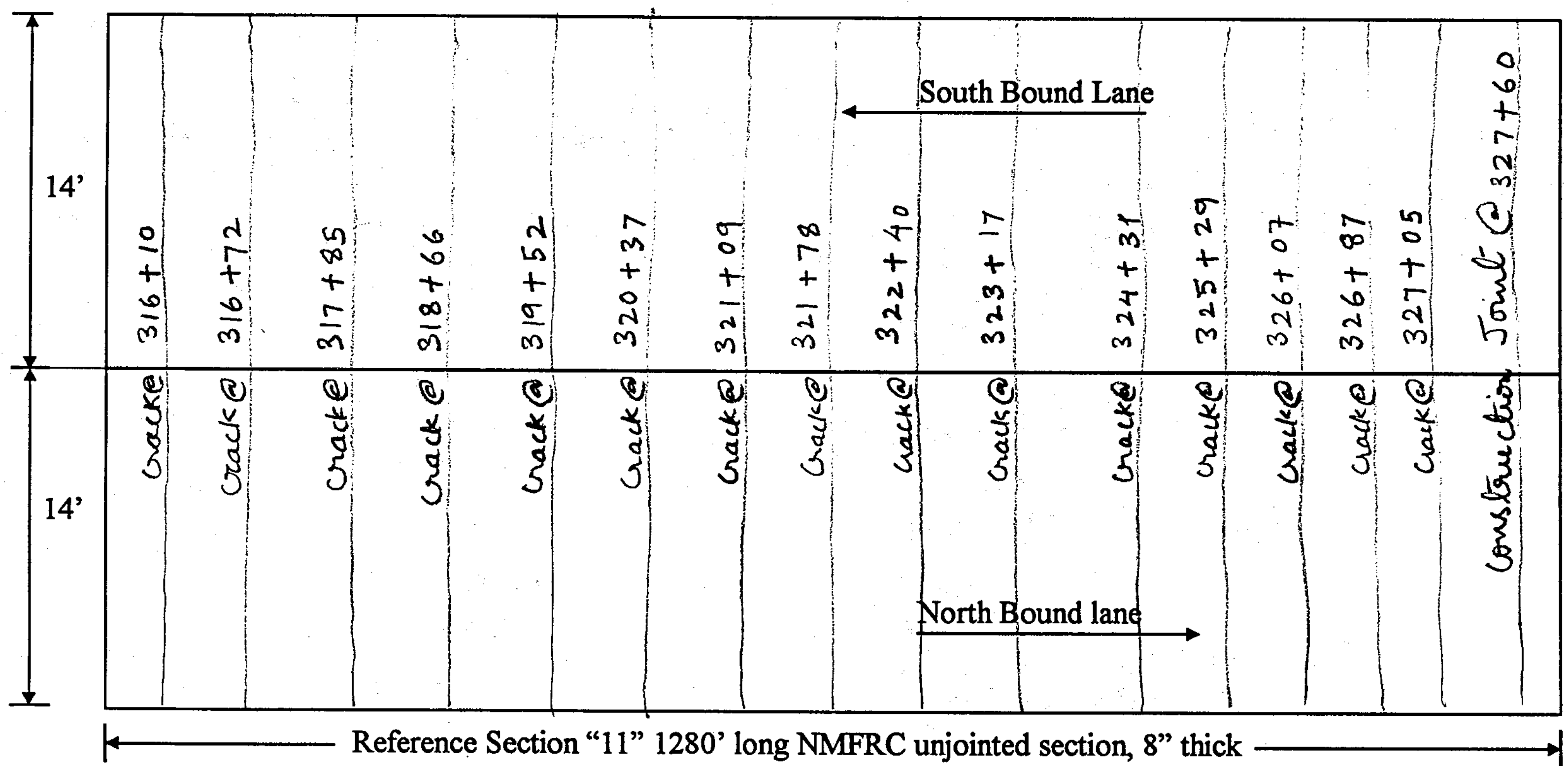


Figure C1 Crack Location for unjointed section of the NMFRC Full-Depth Pavement as observed on April 17, 1998

TABLE C1

Difference in PK nail distances measured across the Cracks in the Unjointed Section for Full Depth Pavement for measurements done on 09/10/96 and 04/17/98

('+' sign indicates increase in distance, '-' sign indicates decrease in distance)

Crack at	North Bound Lane		South Bound Lane	
	East Edge	Center (NB Lane side)	Center (SB Lane side)	West Edge
	(Inches)	(Inches)	(Inches)	(Inches)
326+87	0.1250	0.1250	0.1875	-0.1250
326+07	0.1250	0.0000	0.2500	0.2500
325+29	-0.1250	0.2500	-0.0625	-0.1250
324+31	0.0625	0.1875	0.1250	0.1250
323+17	0.1875	0.1250	0.1875	0.1875
322+40	0.1875	0.1250	0.1875	0.0000
321+78	0.0625	0.1250	0.1250	0.0625
321+09	-0.1250	0.1875	0.1250	0.1875
320+37	0.0625	0.2500	0.2500	-0.2500
319+52	0.0000	-0.2500	0.2500	0.1875
318+66	0.1875	0.0625	0.0625	0.2500
317+85	0.0625	0.1250	0.0000	-0.1875
316+72	0.0625	0.0625	0.2500	0.2500

TABLE C2
Difference in the PK nail measurements for the North Bound Lane
(Full Depth Pavement)
East Edge of North Bound Lane
('+' sign indicates increase in distance and '-' indicates decrease in distance)
The cracks were numbered from North to South

Joint #	Difference in PK nail readings between 08/16/96 and 08/25/96	Difference in PK nail readings between 08/16/96 and 09/07/96	Difference in PK nail readings between 08/16/96 and 10/16/96	Difference in PK nail readings between 08/16/96 and 05/28/97	Difference in PK nail readings between 08/16/96 and 07/28/97	Difference in PK nail readings between 08/16/96 and 11/07/97	Difference in PK nail readings between 08/16/96 and 04/17/98
	(Inches)	(Inches)	(Inches)	(Inches)	(Inches)	(Inches)	(Inches)
1	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
3	0.0625	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
4	0.0625	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625
5	0.0000	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625
6	0.0000	0.0000	0.0625	0.1250	0.0000	0.0625	-0.0625
7	0.0000	0.0625	0.0625	0.1875	0.0625	0.0625	0.0625
8	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
9	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
10	0.0625	0.0625	0.0625	0.0625	0.0625	0.1250	0.1250
11	0.0625	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625
12	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
13	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
14	0.0000	0.0625	0.0625	0.0000	0.0000	0.0000	-0.1250
15	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250	0.1250
16	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
17	0.0625	0.0625	0.1250	0.1250	0.1250	0.1250	0.1250
18	0.0625	0.1250	0.1250	0.1250	0.0625	0.0625	0.0625
19	0.0625	0.0000	0.0000	0.1250	0.0625	0.0625	0.1250
20	0.0000	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
21	0.0625	0.1250	0.1250	0.1250	0.0625	0.1250	0.1875
22	0.0625	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625
23	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
24	0.0625	0.1250	0.1250	0.1250	0.0625	0.1250	0.1250
25	0.0625	0.1250	0.1250	0.1250	0.0625	0.1250	0.1250
26	-0.0625	0.0000	-0.0625	0.0000	0.0000	0.0000	0.0000
27	0.0000	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
28	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
29	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
30	-0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
31	0.0000	0.0625	0.0625	0.1250	0.0625	0.1250	0.1250
32	0.0625	0.0625	0.0000	0.0625	0.0625	0.0625	0.0625
33	0.0625	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
34	0.0625	0.1250	0.1250	0.1875	0.0625	0.1250	0.1250
35	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
36	0.0000	0.0625	0.0625	0.1250	0.0625	0.1250	0.1250
37	0.0000	0.1250	0.1250	0.1875	0.0625	0.0625	0.0625

Table C2 Continued

38	0.1250	0.1875	0.1250	0.1250	0.1875	0.1875	0.1875
39	0.1250	0.1250	0.1250	-0.1875	-0.1875	-0.1250	-0.1250
40	0.1875	0.1875	0.1875	0.1875	0.1875	0.1875	0.1875
41	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
42	0.0000	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
43	0.0625	0.0625	0.1250	0.1250	0.1250	0.1250	0.1250
44	0.0000	0.0625	0.0000	0.0625	0.0625	0.1250	0.1250
45	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250	0.0625
46	0.0000	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625
47	0.0000	0.1250	0.1250	0.0625	0.0625	0.0625	0.0625
48	0.0000	0.0625	0.1250	0.0000	0.0000	0.0000	0.0000
49	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
50	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625	0.0625
51	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
52	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0000
53	0.0000	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
54	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
55	0.0000	0.0000	0.0625	-0.0625	0.0000	0.0000	0.0625
56	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
57	-0.0625	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
58	-0.0625	-0.0625	0.0000	0.0000	0.1875	0.2500	0.2500
59	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625	0.0625
60	0.0625	0.0625	0.0625	0.0625	-0.0625	-0.0625	0.0000
61	0.0625	0.0000	0.0000	0.1250	0.1250	0.1250	0.1250
62	0.0000	0.1250	0.1875	0.1250	0.1250	0.1250	0.1250
63	-0.0625	-0.0625	-0.0625	-0.0625	0.0625	0.0625	0.0625
64	0.0000	0.0625	0.0625	0.0625	0.0000	0.0625	0.0625
65	0.0000	0.0625	0.0625	0.1250	0.1875	0.1875	0.1875
66	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625	0.0625
67	0.0000	0.0625	0.1875	0.0625	0.0000	0.0000	0.0000
68	0.0000	0.0000	0.0625	0.0000	-0.0625	0.0000	0.0000
69	0.0000	0.0000	0.0000	0.0000	0.0625	0.0625	0.0000
70	0.0625	0.1250	0.1250	0.0625	0.1250	0.1250	0.1875
71	0.0625	0.1250	0.1250	0.1250	0.0625	0.0625	0.0625
72	0.0000	0.0000	0.0625	0.0000	0.0625	0.0625	0.0625
73	0.0625	0.1875	0.1875	0.1250	0.0625	0.0625	0.0625
74	0.0000	0.0000	0.0000	0.0625	0.0000	0.0000	0.0625
75	0.0000	0.0625	0.0625	0.1875	0.0625	0.1250	0.1250
76	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250	0.1250
77	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.1250
78	0.0625	0.0625	0.0625	0.0000	0.1250	0.1250	0.1250
79	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.0625
80	0.0625	0.1250	0.1250	-0.0625	-0.0625	-0.0625	-0.0625
81	0.0625	0.1250	0.0625	0.1250	0.1250	0.1250	0.1250
82	0.0000	0.0625	0.0625	0.1250	0.0625	0.1250	0.1250
83	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
84	0.0000	0.0625	0.0000	0.1250	0.0625	0.0625	0.0000
85	0.0000	0.0625	0.0000	0.1250	0.0000	0.0000	0.0000
86	0.0000	0.0000	0.0000	0.0625	0.0000	0.0000	-0.0625
87	0.0000	0.0000	0.0000	0.1250	0.0625	0.0625	0.0625
88	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625

Table C2 Continued

89	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
90	0.0000	-0.0625	0.0000	0.0625	0.0625	0.0625	0.0625
91	0.0625	0.0000	0.0625	0.1250	0.0625	0.0625	0.0625
92	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000	-0.0625
93	0.0000	-0.0625	-0.0625	0.0625	0.0625	0.0625	-0.0625
94	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250
95	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000	0.0000
96	0.0625	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
97	0.0625	0.1250	0.1250	0.1250	0.1250	0.1875	0.1250
98	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
99	0.0000	-0.0625	-0.0625	-0.0625	0.0000	0.0000	0.0000
100	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
101	-0.0625	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0000
102	0.0000	-0.1250	-0.0625	0.1250	0.0000	0.0000	0.0000
103	-0.0625	-0.0625	-0.0625	0.0000	0.0000	0.0000	-0.0625
104	0.0625	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
105	-0.0625	-0.0625	0.0000	0.0000	-0.0625	0.0000	0.0000
106	0.0000	-0.0625	0.0000	0.0000	0.0000	0.0000	-0.0625
107	-0.0625	0.0000	0.0625	0.0000	0.0000	0.0625	0.0625
108	0.0000	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0625
109	-0.0625	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
110	0.0000	0.0625	0.0625	0.0625	0.0000	0.0625	0.0000
111	0.0000	0.0000	0.0625	0.0625	0.0000	0.0000	0.0000
112	-0.0625	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
113	-0.0625	0.0000	0.0000	0.0000	-0.0625	-0.0625	0.0000
114	0.0000	0.0000	0.0625	0.0625	0.0000	0.0000	0.0000
115	0.0000	0.0000	0.0000	0.0625	0.0000	0.0625	0.0000
116	0.0625	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625
117	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250
118	0.0000	0.0625	0.0625	0.0000	0.0625	0.0625	0.0625
119	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0000
120	0.0625	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
121	-0.0625	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0000
122	-0.0625	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0000
123	0.0000	0.0000	0.0625	0.0000	0.0000	0.0625	0.0625
124	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625	0.0000
125	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250
126	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0000
127	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0000
128	0.0625	0.0625	0.0625	0.1250	0.0625	0.1250	0.1250
129	-0.0625	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0000
130	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
131	0.0000	-0.1250	-0.0625	0.0000	-0.0625	-0.0625	-0.0625
132	-0.0625	0.0000	0.0000	-0.0625	-0.0625	0.0000	0.0000
133	0.0000	0.0000	0.0000	0.0625	0.0000	0.0625	0.0625
134	0.0625	0.0625	0.0625	0.1250	0.1250	0.1250	0.1875
135	-0.0625	0.0000	0.0000	0.0000	-0.0625	-0.0625	-0.1250
136	0.0000	0.0000	0.0625	0.0000	0.0000	0.0625	0.0625
137	0.0000	-0.0625	0.0000	0.0000	0.0625	0.1250	0.0625

1 Inch = 25.4 mm

TABLE C3

Difference in the PK nail measurements for the North Bound Lane
(Full Depth Pavement)
West Edge of North Bound Lane
('+' sign indicates increase in distance and '-' sign indicates decrease in distance)
The cracks were numbered from North to South

Joint #	Difference in PK nail readings between 08/16/96 and 08/25/96	Difference in PK nail readings between 08/16/96 and 09/07/96	Difference in PK nail readings between 08/16/96 and 10/16/96	Difference in PK nail readings between 08/16/96 and 05/28/97	Difference in PK nail readings between 08/16/96 and 07/28/97	Difference in PK nail readings between 08/16/96 and 11/07/97	Difference in PK nail readings between 08/16/96 and 04/17/98
	(Inches)	(Inches)	(Inches)	(Inches)	(Inches)	(Inches)	(Inches)
1	0.0000	0.0625	0.0625	0.1250	0.0625	0.1250	0.1875
2	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
3	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
4	0.0000	0.0000	0.0625	0.0625	0.0000	0.0625	0.0625
5	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
6	-0.0625	0.0625	0.1250	0.1250	0.0625	0.1250	0.1250
7	-0.0625	0.0625	0.0625	0.1875	0.1250	0.1250	0.1250
8	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0625	0.0625
9	0.0625	0.1250	0.1250	0.1875	0.1250	0.1250	0.1250
10	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
11	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
12	0.0625	0.0625	0.0625	0.1250	0.1250	0.1250	0.1250
13	0.0000	0.1250	0.1250	0.1875	0.1250	0.1250	0.1250
14	0.0625	0.0625	0.0000	0.0625	0.0625	0.1250	0.0000
15	0.0625	0.0625	0.1250	0.1875	0.0625	0.0625	0.0625
16	0.0000	0.0625	0.0625	0.0625	0.0000	0.0000	0.0625
17	0.0000	0.1250	0.1250	0.1875	0.1250	0.1875	0.1875
18	-0.0625	0.0000	0.0000	0.0000	-0.0625	-0.0625	-0.0625
19	0.0000	0.0625	0.0000	0.0625	0.0625	0.0625	0.0625
20	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
21	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.1250
22	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
23	0.0000	0.0000	0.0000	0.0000	-0.0625	-0.0625	-0.0625
24	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
25	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
26	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
27	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
28	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.1250
29	0.1250	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
30	0.0000	0.1250	0.0625	0.1250	0.0625	0.0625	0.0625
31	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250	0.1250
32	0.0000	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
33	0.0000	0.1250	0.1250	0.0000	0.0000	0.0000	0.0000
34	0.1250	0.1250	0.1250	0.1875	0.1250	0.1250	0.1250
35	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
36	0.0625	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625
37	0.0625	0.1250	0.1250	0.0625	0.1250	0.1250	0.1250
38	0.0000	0.0625	0.0625	0.1250	0.0000	0.0625	0.0625
39	0.0625	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625

Table C3 Continued

40	0.0625	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
41	0.0625	0.1250	0.1250	0.1250	0.0625	0.0625	0.0625
42	0.0000	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
43	0.0000	0.1250	0.1250	0.1250	0.1250	0.1250	0.1250
44	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
45	-0.0625	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
46	0.0625	0.1250	0.1250	0.1875	0.1250	0.1250	0.1250
47	0.0625	0.0625	0.0625	0.1250	0.0625	0.0625	0.1250
48	0.0000	0.0625	0.1250	0.0625	0.0625	0.1250	0.0000
49	0.0000	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625
50	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625
51	0.0625	0.1250	0.1250	0.1250	0.0625	0.0625	0.0625
52	0.0000	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
53	0.0000	0.0625	0.1250	0.0625	0.0625	0.0625	0.0625
54	0.0625	0.1250	0.1250	0.1250	0.0625	0.0625	0.0625
55	0.0000	0.0000	0.0000	0.0000	0.0625	0.0625	0.0625
56	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250	0.1250
57	0.0000	0.0625	0.0625	0.1875	-0.1250	-0.1250	-0.1250
58	0.0000	0.0000	0.0000	0.0625	-0.0625	0.0625	0.0625
59	0.0625	0.1250	0.1250	0.1250	0.1250	0.1250	0.1250
60	0.0000	0.0625	0.0625	0.0625	-0.0625	-0.0625	-0.0625
61	-0.0625	0.0000	0.0000	0.1250	0.1250	0.1250	0.1250
62	0.0000	0.0625	0.1250	0.1875	0.1875	0.2500	0.2500
63	0.0000	-0.0625	0.0000	0.0000	0.0000	0.0000	-0.0625
64	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
65	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625	0.0625
66	0.1250	0.0625	0.1250	0.1250	0.1875	0.1875	0.1875
67	0.1250	0.1250	0.1250	0.1250	0.1250	0.1250	0.1250
68	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
69	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625	0.0000
70	0.0625	0.1250	0.1875	0.1875	0.1875	0.1875	0.1875
71	0.0000	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
72	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
73	0.0625	0.0625	0.0625	0.1250	0.0625	0.0625	0.1250
74	0.0000	0.0625	0.0625	0.0625	0.0000	0.0000	0.0625
75	0.0625	0.0625	0.0625	0.1250	0.1250	0.1250	0.1250
76	0.0625	0.1250	0.1250	0.1250	0.1250	0.1250	0.1250
77	0.0000	0.0000	0.0000	0.0000	-0.0625	-0.1250	-0.1250
78	0.0625	0.1250	0.1250	0.1875	0.0625	0.0625	0.0625
79	0.0000	0.0000	0.0000	0.0000	0.0625	0.0625	0.0625
80	0.0000	0.0625	0.0625	0.0000	0.1250	0.1250	0.1250
81	0.0625	0.0000	0.0000	0.0000	0.0000	0.0000	-0.0625
82	0.0625	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
83	0.0000	0.0625	0.0000	0.0000	0.0625	0.0625	-0.0625
84	0.0625	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625
85	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
86	0.0000	0.0000	0.0000	-0.1250	0.0000	0.0000	0.0000
87	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
88	0.0000	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0000
89	0.0625	-0.0625	0.0000	0.0625	0.0625	0.0625	0.0625
90	0.0000	-0.0625	0.0000	0.1250	0.0625	0.0625	0.0625
91	0.0625	0.0625	0.0625	0.0625	0.0625	0.1250	0.0625

Table C3 Continued

92	0.0625	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625
93	0.0625	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
94	0.0000	-0.0625	0.0000	0.0625	0.0625	0.0625	0.0625
95	0.0000	-0.0625	-0.0625	0.0000	0.0000	0.0000	0.0625
96	0.0625	0.1875	0.0000	0.1250	0.0625	0.1250	0.0625
97	0.0000	0.0000	0.0625	0.0625	0.0625	0.1250	0.1250
98	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
99	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
100	-0.0625	0.0000	0.0000	0.0625	-0.1875	-0.1875	-0.1250
101	0.0000	-0.0625	0.0000	0.0625	0.0000	0.0000	0.0000
102	0.0625	0.0625	0.0625	0.1250	0.0625	0.1250	0.1250
103	-0.0625	-0.0625	-0.0625	0.0000	-0.0625	0.0000	0.0625
104	0.0000	0.1250	0.1250	0.1250	0.1250	0.1250	0.1875
105	0.0000	0.0000	0.0625	0.0625	0.0000	0.0000	-0.0625
106	0.0000	0.0625	0.0625	0.0000	0.0000	0.0000	-0.0625
107	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
108	0.0625	0.0000	0.0625	0.1250	0.0625	0.1250	0.0625
109	0.0000	0.0000	0.0000	0.0000	-0.0625	0.0000	0.0000
110	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.0000
111	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000	-0.0625
112	0.0000	0.1250	0.0625	0.0625	0.0000	0.0000	-0.0625
113	0.0000	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
114	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000	0.0625
115	-0.0625	0.0000	0.0000	0.0000	-0.0625	-0.0625	-0.0625
116	0.0000	0.0000	0.0625	0.0000	0.0000	0.0625	0.1250
117	-0.0625	0.0625	0.0000	0.0000	0.0000	0.0000	0.0625
118	0.0000	0.0625	0.0625	0.0625	-0.0625	-0.0625	0.0000
119	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
120	0.0625	0.0000	0.0625	0.0625	0.1250	0.1250	0.1250
121	-0.0625	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
122	0.0000	-0.0625	0.0000	0.0000	-0.1875	-0.1875	-0.1250
123	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625
124	0.0000	0.0000	0.0000	0.0625	-0.0625	-0.0625	-0.0625
125	-0.0625	-0.0625	0.0000	0.0000	-0.0625	0.0000	0.0625
126	0.0000	0.0000	0.0625	0.0000	0.0625	0.1250	0.0625
127	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625	0.1250
128	0.0000	0.0000	0.0000	0.0625	0.0625	0.0625	0.0000
129	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625	0.1250
130	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0000
131	0.0000	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0625
132	0.0000	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
133	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625
134	0.0000	0.0625	0.0625	0.1250	0.0000	0.0000	-0.0625
135	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625
136	0.0000	0.1250	0.1250	0.0000	0.0000	0.0000	0.0000
137	0.0000	-0.0625	0.0000	0.0000	0.0625	0.0625	0.0625

1 Inch = 25.4 mm

TABLE C4

Difference in the PK nail measurements for the South Bound Lane (Full Depth Pavement)

East Edge of South Bound Lane

('+' indicates increase in distance and '-' sign indicates decrease in distance)

The cracks were numbered from North to South

Jt. #	Difference in PK nail readings between 08/27/96 and 09/07/96 (Inches)	Difference in PK nail readings between 08/27/96 and 10/16/96 (Inches)	Difference in PK nail readings between 08/27/96 and 05/28/97 (inches)	Difference in PK nail readings between 08/27/96 and 07/28/97 (inches)	Difference in PK nail readings between 08/27/96 and 11/07/97 (inches)	Difference in PK nail readings between 08/27/96 and 04/17/98 (inches)
1	0.0625	0.1250	0.1250	0.0000	0.0000	0.0625
2	-0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
3	0.0625	0.1250	0.1250	0.0625	0.0625	0.1250
4	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625
5	0.0625	0.1250	0.0625	0.0625	0.0625	0.0625
6	0.0625	0.1250	0.0625	0.0625	0.0625	0.1250
7	0.0000	0.1250	0.1250	0.0000	0.0000	0.0625
8	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625
9	0.0000	0.0625	0.1250	0.0000	0.0625	0.0625
10	0.0000	0.0000	0.1250	-0.1875	-0.1875	-0.1250
11	0.0000	0.0625	0.0625	-0.0625	0.0000	0.0625
12	-0.0625	-0.0625	0.0000	-0.1250	-0.1250	-0.0625
13	0.0625	0.1875	0.1250	0.0625	0.1250	0.1250
14	0.0625	0.0625	0.0625	0.0625	0.0625	0.1250
15	0.0625	0.1250	0.1250	0.0625	0.1250	0.1875
16	-0.0625	0.0000	0.1875	0.0000	0.0000	0.0000
17	0.0000	0.0625	0.1250	0.0625	0.0625	0.0625
18	0.0000	0.1250	-0.1875	0.0000	0.0000	0.0000
19	0.0000	0.0625	0.0625	0.0625	0.1250	0.1250
20	0.0000	0.0625	0.0625	0.0000	0.0000	0.0000
21	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
22	0.0000	0.0625	0.1250	0.0625	0.1250	0.1250
23	0.0625	0.1250	0.0625	0.0000	0.0000	-0.0625
24	0.0000	0.1250	0.0625	0.0000	0.0000	0.0625
25	0.0625	0.0625	0.1250	0.1250	0.1250	0.1875
26	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625
27	0.0625	0.0625	0.0000	0.0625	0.1250	0.0625
28	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250
29	0.0000	0.0625	0.1250	0.0000	0.0625	0.0625
30	0.0625	0.0625	0.0625	0.0625	0.0625	-0.0625
31	0.1250	0.0625	0.1250	0.0000	0.0000	0.0000
32	0.0625	0.1250	0.0625	0.0625	0.0625	0.0625
33	0.0000	0.0625	0.0625	0.0000	0.0000	0.0000
34	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
35	0.0000	0.0625	0.0000	-0.0625	0.0000	0.0625
36	-0.0625	-0.0625	0.0000	0.0000	0.0000	0.0000
37	0.0000	0.0625	0.0625	0.0000	0.0000	0.0000
38	0.0000	0.0000	0.1250	0.0000	0.0000	0.0625
39	0.0625	0.1250	0.0625	0.0000	0.0000	0.0625

Table C4 Continued

40	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
41	0.0625	0.1250	0.0625	0.0625	0.0625	0.0625
42	0.1250	0.1250	0.0000	0.0000	0.0000	0.0000
43	0.1250	0.1250	0.1250	0.0625	0.0625	0.0625
44	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625
45	-0.0625	0.0000	0.0625	0.0000	0.0000	0.0000
46	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625
47	0.0000	0.0000	0.0625	0.0625	0.0625	0.1250
48	0.0000	0.0625	0.0625	0.0625	0.1250	0.1250
49	0.0625	0.0625	0.0625	0.0000	0.0625	0.0625
50	0.0000	0.0625	0.1250	0.0625	0.1250	0.1250
51	-0.0625	-0.0625	0.0000	-0.0625	0.0000	0.0000
52	-0.0625	-0.0625	0.0625	0.1250	0.1250	0.0625
53	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
54	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
55	0.1250	0.1250	0.0000	0.0000	0.0000	0.0000
56	0.0000	0.0000	0.0625	0.0000	0.0000	0.0625
57	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
58	0.0625	0.1250	0.0625	0.0625	0.0625	0.1250
59	0.0000	0.0000	0.0625	0.0625	0.0625	0.1250
60	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625
61	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625
62	0.0000	-0.0625	0.1250	0.0625	0.1250	0.1250
63	0.0000	0.0625	0.0000	0.0000	0.0000	0.0625
64	0.0000	0.0000	-0.0625	0.0000	0.0000	0.0000
65	0.0625	0.0625	0.0625	0.0625	0.0625	0.1250
66	0.0000	0.0000	0.1250	0.0625	0.0625	0.0625
67	0.0000	0.1250	0.0000	0.0000	0.0000	0.0000
68	-0.0625	-0.0625	0.0000	-0.0625	-0.0625	-0.0625
69	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
70	-0.0625	-0.0625	0.0625	0.0000	0.0000	0.0625
71	0.0000	0.0000	0.1250	0.0000	0.0000	0.0625
72	0.0625	0.0625	0.0625	0.0625	0.1250	0.1250
73	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625
74	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
75	0.0000	0.0625	0.0625	0.0000	0.0625	0.0625
76	0.0625	0.0625	-0.0625	0.0625	0.0625	0.0625
77	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
78	0.0000	0.0625	0.1250	0.0625	0.1250	0.1250
79	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
80	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
81	0.0625	0.1250	0.0625	0.0625	0.0625	0.0625
82	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
83	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
84	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625
85	0.0000	0.0625	0.0000	-0.1250	-0.0625	0.0000
86	0.0000	0.0625	0.0000	0.0000	-0.0625	-0.0625
87	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
88	-0.0625	0.0000	0.0000	0.0000	0.0000	0.0000
89	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
90	0.0000	0.0000	0.0625	0.0000	0.0000	0.0625

Table C4 Continued

91	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625
92	-0.0625	-0.0625	0.0000	-0.0625	-0.0625	-0.0625
93	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
94	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
95	0.0000	0.0000	0.0625	0.0000	0.0000	0.0625
96	0.0625	0.0000	0.0625	0.0625	0.0625	0.0625
97	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
98	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625
99	0.0000	0.0625	0.0000	0.0625	0.0625	0.0625
100	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625
101	0.0625	0.0625	0.0000	0.0625	0.0625	0.0625
102	0.0000	0.0000	-0.0625	0.0000	0.0000	0.0000
103	0.0000	0.0000	-0.1875	0.0625	0.0625	0.0625
104	0.0000	0.0625	-0.0625	0.0625	0.0625	0.0000
105	0.0625	0.0000	0.0000	0.0625	0.0625	0.0625
106	-0.0625	-0.0625	-0.1875	-0.0625	-0.0625	-0.0625
107	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
108	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625
109	0.0625	0.1250	0.0625	0.0000	0.0000	0.0625
110	-0.0625	0.0000	-0.0625	-0.0625	-0.0625	-0.0625
111	0.0625	0.0625	-0.0625	0.0000	0.0000	0.0625
112	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625
113	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
114	0.0625	0.0625	-0.0625	0.0625	0.0625	0.0625
115	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
116	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625
117	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
118	0.1250	0.1250	0.0000	0.0625	0.0625	0.0625
119	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
120	0.0625	0.0625	-0.0625	0.0625	0.0625	0.0000
121	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
122	0.0625	0.1250	0.1250	0.1250	0.1250	0.1875
123	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
124	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
125	0.1250	0.1250	0.1250	0.0000	0.0000	0.0625
126	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
127	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
128	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
129	0.0000	0.0625	0.0625	0.0625	0.0625	0.0000
130	0.0625	0.0625	0.0625	0.0000	0.0000	0.0625
131	0.0000	0.0625	0.0625	-0.0625	0.0000	0.0000
132	0.0625	0.0625	0.0625	0.0000	0.0000	0.0625
133	0.0000	0.0625	0.0625	0.0000	-0.0625	-0.0625
134	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
135	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
136	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
137	0.0000	0.0625	0.0625	0.0000	0.0625	0.0625

1 Inch = 25.4 mm

TABLE C5

Difference in the PK nail measurements for the South Bound Lane (Full Depth Pavement)

West Edge of South Bound Lane

('+' indicates increase in distance and '-' sign indicates decrease in distance)

The cracks were numbered from North to South

Jt. #	Difference in PK nail readings between 08/27/96 and 09/07/96 (Inches)	Difference in PK nail readings between 08/27/96 and 10/16/96 (Inches)	Difference in PK nail readings between 08/27/96 and 05/28/97 (Inches)	Difference in PK nail readings between 08/27/96 and 07/28/97 (Inches)	Difference in PK nail readings between 08/27/96 and 11/07/97 (Inches)	Difference in PK nail readings between 08/27/96 and 04/17/98 (Inches)
1	0.0625	0.1875	0.1250	0.1250	0.1250	0.1250
2	0.0000	0.0000	0.0000	0.0625	0.0625	0.1250
3	0.0625	0.1250	0.0625	0.0625	0.0625	0.1250
4	0.0625	0.1250	0.1250	0.0625	0.1250	0.1250
5	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
6	0.0625	0.1250	0.0000	0.0000	0.0000	0.0625
7	0.0000	0.1250	0.0625	0.0625	0.0625	0.1250
8	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
9	0.0625	0.1250	0.1250	0.1250	0.1250	0.1250
10	0.0625	0.1250	0.0625	0.1250	0.1250	0.1250
11	0.1250	0.1875	0.1250	0.0625	0.0625	0.1250
12	0.0000	0.0625	0.0625	0.1875	0.1875	0.1250
13	0.0625	0.1250	0.1250	0.0625	0.0625	0.1250
14	0.0000	0.1250	0.1250	0.1250	0.1250	0.1250
15	0.0625	0.1250	0.1250	0.0625	0.0625	0.1250
16	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
17	0.0625	0.1250	0.1250	0.0625	0.0625	0.0625
18	0.0000	0.0625	0.0000	0.0000	0.0625	0.0625
19	0.0000	0.1250	0.0625	0.0625	0.0625	0.0000
20	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
21	0.0625	0.1875	0.0625	0.0000	0.0000	0.0625
22	0.0000	0.0625	0.1250	0.0625	0.1250	0.1250
23	0.0625	0.1250	0.0625	0.0000	0.0000	0.0625
24	0.1250	0.0625	0.1875	0.1250	0.1250	0.1875
25	0.0625	0.0625	0.0625	0.0000	0.0000	0.0625
26	0.0625	0.1250	0.0625	0.0625	0.1250	0.0625
27	0.0000	0.1250	0.1250	0.1250	0.1250	0.1250
28	0.1250	0.1875	0.1875	0.1250	0.1250	0.1875
29	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
30	0.0625	0.1250	0.1250	0.1250	~	0.1875
31	0.0000	0.1250	~	~	~	~
32	0.0625	0.1250	0.0625	0.0625	0.0625	0.0625
33	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625
34	0.0625	0.1250	0.1250	0.1250	0.1250	0.1250
35	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
36	0.0625	0.1250	0.1250	0.1250	0.1250	0.1250
37	0.0625	0.1250	0.0000	0.0625	0.1250	0.1250
38	0.0625	0.0625	0.0625	0.0625	0.0625	0.1250
39	0.0625	0.1250	0.0625	0.0000	0.0000	0.0625

Table C5 Continued

40	0.0000	0.0625	0.0625	0.0000	0.0000	0.0000
41	0.0000	0.0625	0.0625	0.0000	0.0625	0.0625
42	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
43	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
44	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
45	0.0625	0.0625	0.0625	0.1250	0.1250	0.0625
46	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
47	0.0000	0.0625	0.1250	~	~	~
48	0.0000	0.0000	0.1250	0.0625	0.0625	0.1250
49	0.0625	0.1250	0.0625	0.0625	0.0625	0.0625
50	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625
51	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
52	-0.0625	-0.0625	0.0000	0.0000	0.0000	0.0625
53	0.0000	0.0000	0.0000	0.0000	0.0000	0.0625
54	-0.0625	-0.0625	0.0000	0.0000	0.0625	0.0000
55	0.0000	0.0000	0.1250	0.0625	0.0625	0.0625
56	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
57	0.0000	0.0000	0.0625	0.0000	0.0000	0.0000
58	0.0625	0.0625	0.0625	0.0625	0.0625	0.1250
59	0.0000	0.0000	0.0625	0.0625	0.1250	0.1250
60	0.0625	0.0625	0.0000	0.0625	0.0000	0.0625
61	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
62	0.0625	0.0000	0.0625	0.0625	0.1250	0.1250
63	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
64	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
65	0.0625	0.1250	0.0625	0.0000	0.0625	0.0625
66	0.0625	0.0000	0.1875	0.0625	0.0625	0.0625
67	0.0625	0.1250	0.0625	0.0625	0.1250	0.1250
68	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
69	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
70	0.0000	0.0625	0.0625	0.0000	0.0625	0.1250
71	0.0000	0.0625	0.0625	0.0000	0.0625	0.1250
72	0.0625	0.1250	0.0625	0.0625	0.0625	0.1250
73	0.0000	0.0625	0.0000	0.0000	0.0625	0.1250
74	0.0625	0.0625	0.1250	0.0625	0.0625	0.1250
75	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625
76	0.0000	0.0625	0.1250	0.1250	0.1250	0.1875
77	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
78	0.0625	0.0625	0.0625	0.0000	0.0000	0.0625
79	0.0625	0.1250	0.0625	0.0625	0.0625	0.0000
80	0.0000	0.0000	0.0625	0.0000	0.0000	0.0625
81	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250
82	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
83	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
84	0.0625	0.0625	0.1250	0.0625	0.0625	0.1250
85	0.0000	0.1250	0.0625	0.0625	0.0625	0.0625
86	0.0625	0.0625	0.0000	0.0000	0.0625	0.0625
87	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625
88	0.0625	0.1250	0.0625	0.0000	0.0000	0.0000
89	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
90	0.0000	0.0625	0.0625	0.0000	0.0000	0.0625

Table C5 Continued

91	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
92	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
93	0.0000	0.0000	0.1875	0.0000	0.0625	0.0625
94	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
95	0.0000	0.0625	0.0625	0.0625	0.0625	0.0625
96	0.0000	0.0000	0.0625	0.0625	0.0625	0.0625
97	0.0625	0.0000	0.0625	0.0625	0.0625	0.0625
98	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
99	0.0625	0.0625	0.0000	0.0000	0.0625	0.0625
100	0.0625	0.0625	0.0000	0.0625	0.0625	0.0625
101	0.0625	0.0625	0.0625	0.0000	0.0625	0.0625
102	0.0625	0.0625	0.0625	-0.0625	-0.0625	-0.0625
103	0.0000	0.0000	0.0000	0.0000	0.0625	0.0625
104	0.0000	0.0625	0.0000	0.0000	0.0625	0.0625
105	0.0625	0.0625	0.0625	0.0000	0.0625	0.0000
106	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
107	0.0000	0.0625	0.0000	0.0000	0.0000	0.0000
108	0.0625	0.0625	-0.1250	0.0625	0.0625	0.1250
109	0.0000	0.0625	0.0625	0.0625	0.0625	0.1250
110	0.0625	0.0000	0.0000	-0.0625	-0.0625	-0.0625
111	0.0625	0.0625	0.0625	0.0000	0.0000	0.0625
112	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
113	0.0625	0.0625	-0.1250	0.0000	0.0000	0.0000
114	0.0625	0.0625	-0.0625	0.0000	0.0000	0.0000
115	0.0000	0.0625	0.0625	0.0000	0.0625	0.0625
116	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
117	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
118	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
119	0.0625	0.0625	0.1250	0.0625	0.0625	0.0625
120	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
121	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
122	0.0625	0.0625	0.0625	0.0000	0.0000	0.0000
123	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
124	0.0000	0.0000	0.0000	0.0000	0.0625	0.0625
125	0.0000	0.0625	0.0625	0.0000	0.0625	0.0625
126	0.0000	0.0000	0.0000	0.0000	0.0625	0.0625
127	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
128	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
129	0.0625	0.0625	0.0000	0.0000	0.0000	0.0000
130	0.0625	0.1250	0.0000	0.0625	0.0625	0.0625
131	0.0000	0.0625	0.0000	0.0000	0.0625	0.0625
132	0.0625	0.0625	-0.0625	0.0000	0.0000	0.0000
133	0.1250	0.1250	0.0625	0.0625	0.0625	0.0625
134	0.1250	0.1250	0.0000	0.0000	0.0000	0.0000
135	0.0000	0.0000	0.0000	-0.0625	-0.0625	-0.0625
136	0.0000	0.0000	0.0625	0.0625	0.0625	0.1250
137	0.0625	0.0625	-0.1250	0.0625	0.0625	0.0625

1 Inch = 25.4 mm

APPENDIX D

Details of Core Testing

Table D1: Cores on NMFRC Full Depth Pavement Research Project
US 83

North Bound Lane

Sp. #	Location	Diameter (Inches)	Height (Inches)	Remarks (By Inspection)
1	292+86, 7.0' from edge	4.020	6.438	About 5 % Honey Combing Good fiber distribution
		4.020	6.375	
		4.020	6.438	
			6.500	
	Average	**4.020**	**6.438**	
2	298+76, 10.1' from edge	4.020	7.875	About 10 % Honey Combing Good fiber distribution
		4.020	7.938	
		4.020	8.000	
			7.938	
	Average	**4.020**	**7.938**	
3	318+32, 6.6' from edge	4.020	7.781	About 8 % Honey Combing Good fiber distribution
		4.020	7.750	
		4.020	7.781	
			7.719	
	Average	**4.020**	**7.758**	
4	320+44, 10.2' from edge	4.020	8.031	About 8 % Honey Combing Good fiber distribution
		4.020	8.938	
		4.020	8.000	
			8.000	
	Average	**4.020**	**8.242**	

Conversion Table:
1 Inch = 25.4 mm

Table D2: Cores on NMFRC Full Depth Pavement Research Project
US 83 South Bound Lane

Sp. #	Location	Diameter (Inches)	Height (Inches)	Remarks
1	291+70, 5.6' from edge	4.020	6.460	About 10 % Honey Combing Good fiber distribution
		4.020	6.475	
		4.020	6.475	
			6.435	
	Average	**4.020**	**6.461**	
2	294+00, 7.9' from edge	4.020	6.422	About 8 % Honey Combing Good fiber distribution
		4.020	6.475	
		4.020	6.425	
			6.450	
	Average	**4.020**	**6.443**	
3	297+55, 4.3' from edge	4.020	7.906	About 7 % Honey Combing Good fiber distribution
		4.020	7.844	
		4.020	7.906	
			8.000	
	Average	**4.020**	**7.914**	
4	306+02, 8.1' from edge	4.020	7.719	About 10 % Honey Combing Good fiber distribution
		4.020	7.750	
		4.020	7.625	
			7.688	
	Average	**4.020**	**7.696**	
5	312+47, 2.4' from edge	4.020	7.781	About 7 % Honey Combing Good fiber distribution
		4.020	7.750	
		4.020	7.781	
			7.719	
	Average	**4.020**	**7.758**	
6	318+74, 8.4' from edge	4.020	7.813	About 5 % Honey Combing Good fiber distribution
		4.020	7.844	
		4.020	7.781	
			7.875	
	Average	**4.020**	**7.828**	

Conversion Table:
1 Inch = 25.4 mm

Table D3: Testing of Cores on NMFRC Full Depth Pavement Research Project for Compressive Strength
US 83

North Bound Lane

Sp. #	Location	Diameter (Inches)	Height after trimming (Inches)	Area of cross-section (Sq. Inches)	l/d ratio, correction factor *	Corrected Compressive Strength (psi)
1	292+86,	4.020	6.380			
	7.0' from	4.020	6.425	12.692	1.59	5145
	edge	4.020	6.395		Corr. fact.	
			6.412		0.9672	
	Average	**4.020**	**6.403**			
2	298+76,	4.020	7.800			
	10.1' from	4.020	7.827	12.692	1.95	4590
	edge	4.020	7.883		Corr. Fact:	
			7.842		0.9960	
	Average	**4.020**	**7.838**			
3	318+32,	4.020	7.110			
	6.6' from	4.020	7.142	12.692	1.77	4795
	edge	4.020	7.137		Corr. Fact.	
			7.123		0.9816	
	Average	**4.020**	**7.128**			
4	320+44,	4.020	7.915			
	10.2' from	4.020	8.000	12.692	1.98	4090
	edge	4.020	7.975		Corr. Fact:	
			7.948		0.9984	
	Average	**4.020**	**7.960**			

* Correction Factor as per ASTM C42
Conversion Table:
1 Inch = 25.4 mm

Table D4: Testing of Cores on NMFRC Full Depth Pavement Research Project for Compressive Strength US 83 (South Bound Lane)

Sp. #	Location	Diameter (Inches)	Height after trimming (Inches)	Area of cross-section (Sq. Inches)	l/d ratio, correction factor *	Corrected Compressive Strength (psi)
1	291+70,	4.020	6.165			
	5.6' from	4.020	6.165	12.692	1.53	4400
	edge	4.020	6.165		Corr. fact.	
			6.165		0.9620	
	Average	**4.020**	**6.165**			
2	294+00,	4.020	6.345			
	7.9' from	4.020	6.350	12.692	1.58	4115
	edge	4.020	6.350		Corr. Fact.	
			6.345		0.9664	
	Average	**4.020**	**6.348**			
3	297+55,	4.020	7.700			
	4.3' from	4.020	7.700	12.692	1.91	4695
	edge	4.020	7.675		Corr. Fact:	
			7.670		0.9928	
	Average	**4.020**	**7.680**			
4	306+02,	4.020	7.590			
	8.1' from	4.020	7.522	12.692	1.88	4445
	edge	4.020	7.530		Corr. Fact:	
			7.522		0.9904	
	Average	**4.020**	**7.541**			
5	312+47,	4.020	7.560			
	2.4' from	4.020	7.580	12.692	1.88	4055
	edge	4.020	7.535		Corr. Fact.	
			7.512		0.9904	
	Average	**4.020**	**7.547**			
6	318+74,	4.020	7.785			
	8.4' from	4.020	7.750	12.692	1.93	4230
	edge	4.020	7.700		Corr. Fact:	
			7.740		0.9944	
	Average	**4.020**	**7.744**			

* Correction Factor as per ASTM C42
Conversion Table: 1 Inch = 25.4 mm

APPENDIX E

Details of Maturity Testing

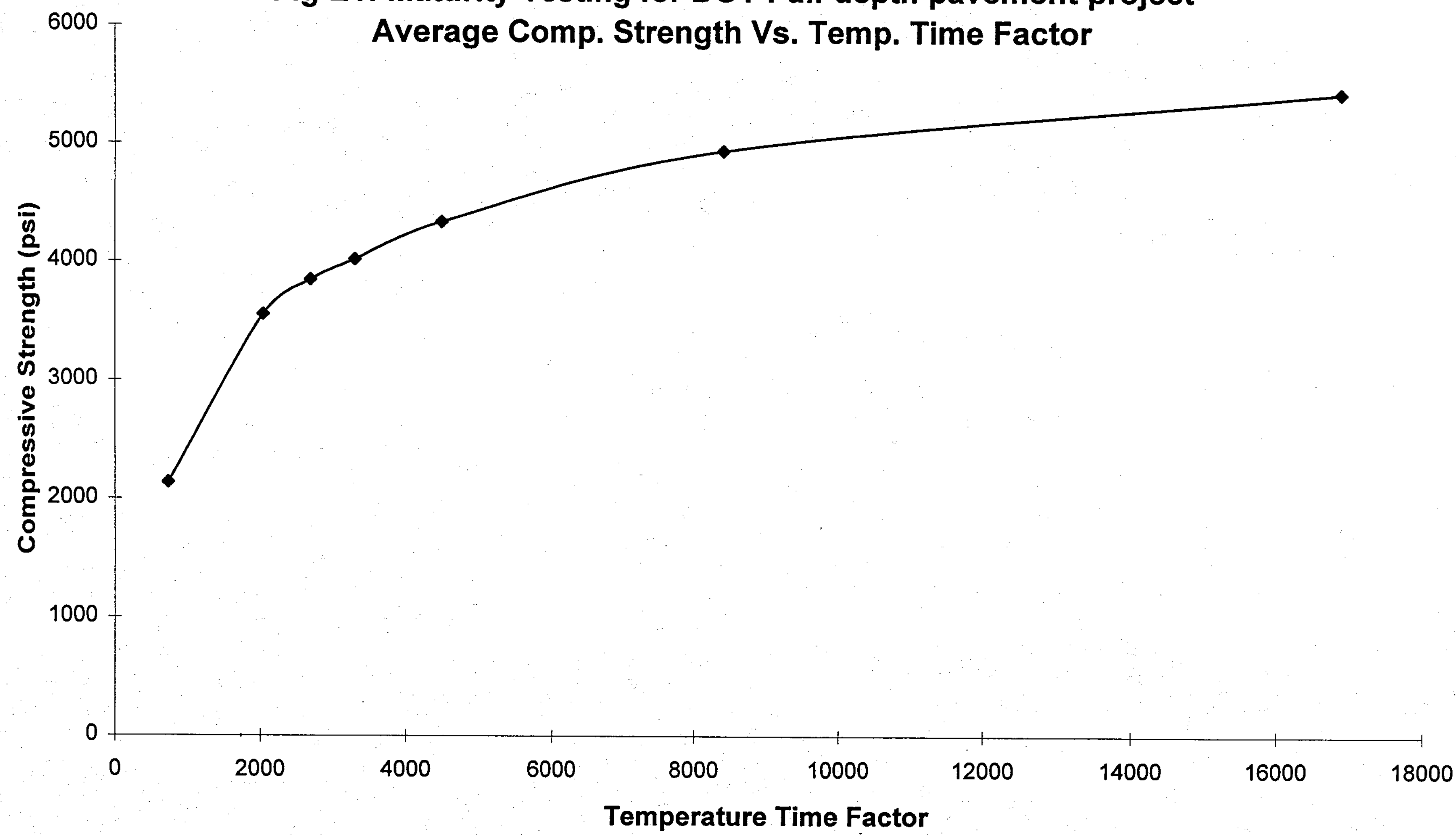
Table E1: Maturity Testing for DOT Project

Mix Design

Day	Date	Sp. Nos.	Comp. Strength	Maturity Meter Readings				
			(psi)	Outlet	Hours	Internal Temp.	Age (hours)	TTF
1	7/3/96	DOT-T4-C1	2015	1	24	30	44.9	745
		DOT-T4-C2	2265	2	24	29	41	708
		Average	**2140**					**727**
3	7/5/96	DOT-T4-C3	3620	1	72	25	109	1953
		DOT-T4-C4	3485	2	72	28	123	2124
		Average	**3555**					**2039**
4	7/6/96	DOT-T4-C5	3885	1	96	25	141	2559
		DOT-T4-C6	3800	2	96	27	164	2832
		Average	**3845**					**2696**
5	7/7/96	DOT-T4-C7	3895	1	120	23	172	3143
		DOT-T4-C8	4130	2	120	25	199	3473
		Average	**4015**					**3308**
7	7/9/96	DOT-T4-C9	4265	1	168	22	229	4245
		DOT-T4-C10	4395	2	168	25	269	4747
		Average	**4330**					**4496**
14	7/16/96	DOT-T4-C11	5035	1	336	22	411	7831
		DOT-T4-C12	4835	2	336	27	499	9019
		Average	**4935**					**8425**
28	7/30/96	DOT-T4-C13	5425	1	672	23	814	15604
		DOT-T4-C14	5420	2	672	27	1012	18229
		Average	**5425**					**16917**

* T4 was a batch during the mix design process.

Fig E1: Maturity Testing for DOT Full-depth pavement project Average Comp. Strength Vs. Temp. Time Factor



APPENDIX F

Results From Tests Conducted By SDDOT

F2

LIFE-CYCLE COST ESTIMATING WORKSHEET

Enter Initial Analysis Year	1999	Project Identification	
Enter Analysis Period	40		
Enter Annual Discount Rate, %	4.06		

		Alternative 1	Alternative 2
		Project Description:	Project Description:
		PJCP	FRCP

Initial Costs Item No.	Item Description	Analysis Year	Calendar Year	Alt 1 Estimated Cost	Alt 1 Present Worth	Alt 2 Estimated Cost	Alt 2 Present Worth
1	PJCP	0	1999	$252,150	$252,150		
2	FRCP	0	1999			$474,730	$474,730
3							
4							
5							
6							
7							
8							
Total Present Worth of Initial Costs				$252,150	$252,150	$474,730	$474,730

Periodic Costs Item No.	Item Description	Analysis Year	Calendar Year	Alt 1 Estimated Cost	Alt 1 Present Worth	Alt 2 Estimated Cost	Alt 2 Present Worth
1	Minor Joint & Spall Repair	18	2017	$20,000	$9,771		
2	Major Joint & Spall Repair	32	2031	$80,000	$22,388		
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
Total Present Worth of Periodic Costs					$32,158		$0

Annual Costs Item No.	Item Description	First Yr. of Ann. Costs Analysis Yr.	First Yr. of Ann. Costs Cal Yr	Last Yr. of Ann. Costs Analysis Yr	Last Yr. of Ann. Costs Cal Yr	Alt 1 Estimated Annual Cost	Alt 1 Present Worth	Alt 2 Estimated Annual Cost	Alt 2 Present Worth
1	Maint Activity for Alt 1	1	2000	40	2039	$580	$11,378		
2									
3									
4									
5									
6									
7									
8									
Total Present Worth of Annual Costs							$11,378		$0

Replacement/Salvage Value Item No.	Item Description	Analysis Year	Calendar Year	Alt 1 Estimated Value	Alt 1 Present Worth	Alt 2 Estimated Value	Alt 2 Present Worth
1							
2							
3						-	
4							
Total Present Worth of Replacement/Salvage Value					$0		$0

TOTAL LCC	Alternative 1	Alternative 2
Present Worth LCC	$295,686	$474,730
Equivalent Uniform Annual LCC	$15,073	$24,200

Lowest LCC Alternative	Alternative 1	
PW Cost Difference From Lowest LCC Alternative	$0	$179,044
% Difference From Lowest LCC Alternative	0	61

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LIFE-CYCLE COST ESTIMATING WORKSHEET

Enter Initial Analysis Year	1999	Project Identification	
Enter Analysis Period	60		
Enter Annual Discount Rate, %	4.06		

	Alternative 1	Alternative 2
Project Description:	PJCP	FRCP

Initial Costs Item No.	Item Description	Analysis Year	Calendar Year	Alt 1 Estimated Cost	Alt 1 Present Worth	Alt 2 Estimated Cost	Alt 2 Present Worth
1	PJCP	0	1999	$252,150	$252,150		
2	FRCP	0	1999			$474,730	$474,730
3							
4							
5							
6							
7							
8							
Total Present Worth of Initial Costs				$252,150	$252,150	$474,730	$474,730

Periodic Costs Item No.	Item Description	Analysis Year	Calendar Year	Alt 1 Estimated Cost	Alt 1 Present Worth	Alt 2 Estimated Cost	Alt 2 Present Worth
1	Minor Joint & Spall Repair	18	2017	$20,000	$9,771		
2	Major Joint & Spall Repair	32	2031	$80,000	$22,388		
3	AC Overlay	40	2039	$200,000	$40,708		
4	Mill & AC Overlay	50	2049	$150,000	$20,507		
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
Total Present Worth of Periodic Costs					$93,373		$0

Annual Costs Item No.	Item Description	First Yr. of Ann. Costs Analysis Yr.	First Yr. Cal Yr	Last Yr. of Ann. Costs Analysis Yr	Last Yr. Cal Yr	Alt 1 Estimated Annual Cost	Alt 1 Present Worth	Alt 2 Estimated Annual Cost	Alt 2 Present Worth
1	Maint Activity for Alt 1	1	2000	40	2039	$580	$11,378		
2	Maint Activity for Alt 1	41	2040	60	2059	$2,000	$5,503		
3									
4									
5									
6									
7									
8									
Total Present Worth of Annual Costs							$16,881		$0

Replacement/Salvage Value Item No.	Item Description	Analysis Year	Calendar Year	Alt 1 Estimated Value	Alt 1 Present Worth	Alt 2 Estimated Value	Alt 2 Present Worth
1							
2							
3							
4							
Total Present Worth of Replacement/Salvage Value					$0		$0

TOTAL LCC	Alternative 1	Alternative 2
Present Worth LCC	$362,404	$474,730
Equivalent Uniform Annual LCC	$16,201	$21,223
Lowest LCC Alternative	Alternative 1	
PW Cost Difference From Lowest LCC Alternative	$0	$112,326
% Difference From Lowest LCC Alternative	0	31

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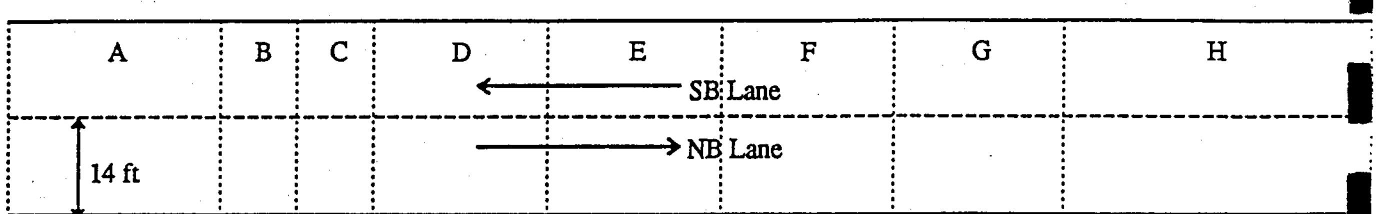
4/11/97

FALLING WEIGHT DEFLECTOMETER RESULTS
US 83 Polyolefin Fiber Concrete Research Site

ITEM	Section A	Section B	Section C	Section D	Section E	Section F	Section G	Section H
Load Transfer (Ave %)	97.0	82.2	73.9	92.1	90.9	84.5	79.9	65.7
Delta Deflections (Mils)	0.28	0.72	1.12	0.34	0.35	0.57	0.75	1.58
K/Kc at X.22 & .31 (Average)	0.99	0.98	0.98	0.98	1.00	0.99	0.96	0.85
K/Kc at X.21 (Average)	1.00	1.00	0.99	0.99	1.01	1.00	1.00	0.98
Elastic Modulus Concrete (Ksi)	4294	3624	3699	3623	3686	3693	3617	3627
Subgrade Properties E Field CBR (FWD)	27.2 6.0	26.9 6.0	27.4 6.1	26.8 6.0	27.2 6.0	27.3 6.1	27.6 6.1	26.5 5.9

These results should be considered with all other testing for this project.

Pavement Test Section Layout (Near MRM 145)



Test Section	Length (ft)	Type	Thickness (in)	Doweled	Joint Spacing (ft)	Volume of Fiber Concrete (yd^3)
A	1000	Plain PCCP Control	8	Yes	20	0
B	250	Fiber (25 lbs/yd^3)	6.5	No	25	173
C	245	Fiber (25 lbs/yd^3)	6.5	No	35	170
D	500	Fiber (25 lbs/yd^3)	8	Yes	25	346
E	490	Fiber (25 lbs/yd^3)	8	Yes	35	339
F	500	Fiber (25 lbs/yd^3)	8	No	25	346
G	490	Fiber (25 lbs/yd^3)	8	No	35	339
H	1290	Fiber (25 lbs/yd^3)	8	No	None	892
					Total	2,605

Definitions of Falling Weight Deflectometer Terms

1. Load Transfer is the average percentage of a normalized 9,000 lbs load that is transferred across a transverse joint in PCC pavement. The Load Transfer value is considered to be of concern when it reaches less than 80%.

2. Delta Deflections is the differences in the FWD recorded movement (mils 1/1000 of a inch) from one side of the transverse joint to the other. The Delta Deflection value is considered to be of concern when it reaches around 7 to 8 mils.

3. K/Kc at test locations X.22 and X.31 is the ratio of the concrete strength at the joint versus the strength of the concrete at the center of the slab (X.10). The drawing shown below shows the FWD test locations for each concrete slab. This K/Kc ratio is considered to be of concern when it is less than 0.70. This may indicate a significant loss of strength of the concrete near the joint.

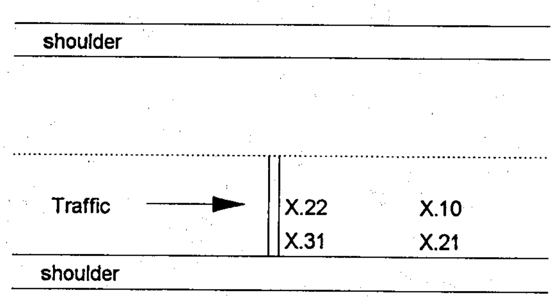
4. K/Kc at test location X.21 is the ratio of the concrete strength at the edge of the slab versus the strength of the concrete at the center of the slab (X.10). The drawing shown below shows the FWD test locations for each concrete slab. The K/Kc at X.21 is considered to be of concern when it reaches less than 0.80. This may indicate a significant loss of strength of the concrete near the edge.

5. Elastic Modulus values are backcalculated using a system of equations which include data such as, ambient temperature, pavement structure thickness, soil type, etc. Some of the coefficients used in the system of equations to backcalculate the Elastic Modulus are assuming that the concrete is plain concrete. These coefficients may differ for polyolefin fiber concrete which may account for the lower elastic modulus values on the fiber sections. New PCC pavements should have elastic modulus values around 4,000 Ksi.

6. (E Field) is the FWD calculated Field Elastic Modulus of the top 20 feet of subgrade.

7. (CBR -FWD) is the California Bearing Ratio of the top 20 feet of subgrade backcalculated from FWD data.

TEST PATTERN



X is the test panel number (1,2,3 etc)

Roughness Index Tests Conducted by SDDOT

Tested 11/6/1996					
	Cls/Hwy/Sfx 1083 SB		DLW 221	aran v. 4.28 sride v. 1.14a 2 Foot Sensivity	
MRM	Disp (miles)	Driver IRI (m/km)	Passenger IRI (m/km)	Driver SDI	Passenger SDI
144	0.95	1.07	1.09	4.85	4.90
144	0.90	1.15	1.10	4.64	4.61
144	0.85	1.09	1.06	4.81	4.80
144	0.80	1.21	0.92	4.73	4.92
144	0.75	1.25	1.08	4.74	4.74
144	0.70	1.43	1.14	4.66	4.83
144	0.65	1.36	1.11	4.74	4.75
144	0.60	1.26	1.16	4.86	4.78
144	0.55	1.07	1.05	4.91	4.88
144	0.50	1.45	1.13	4.86	4.85
144	0.45	1.30	1.49	4.82	4.53
144	0.40	1.29	1.06	4.76	4.85
144	0.35	1.14	1.23	4.72	4.76
144	0.30	1.53	1.15	4.57	4.78
144	0.25	1.14	1.04	4.86	4.85
144	0.20	1.18	0.98	4.80	4.78
144	0.15	1.04	1.03	4.82	4.89
144	0.10	1.19	1.04	4.90	4.95
144	0.05	1.41	1.07	4.74	4.90
144	0.00	1.32	0.95	4.87	4.92

Note: IRI - International Roughness Index
SDI - South Dakota Index
IRI - New Paving 1.2 or less
SDI - Range 5 to 0
5 is good . New paving should be 4.4 or greater.

Roughness Index Tests Conducted by SDDOT

Tested 11/6/1996					
	Cls/Hwy/Sfx 1083 NB		DLW 111	aran v. 4.28 sride v. 1.14a 2 Foot Sensivity	
MRM	Disp (miles)	Driver IRI (m/km)	Passenger IRI (m/km)	Driver SDI	Passenger SDI
144	0.00	1.21	1.18	4.79	4.81
144	0.05	1.17	1.17	4.73	4.82
144	0.10	1.13	1.04	4.79	4.82
144	0.15	1.12	0.94	4.62	4.84
144	0.20	1.28	1.25	4.47	4.56
144	0.25	1.16	1.01	4.82	4.84
144	0.30	1.14	0.91	4.76	4.79
144	0.35	1.14	1.07	4.73	4.78
144	0.40	1.21	1.11	4.72	4.75
144	0.45	1.10	0.88	4.75	4.89
144	0.50	1.16	0.92	4.78	4.89
144	0.55	1.14	1.10	4.74	4.73
144	0.60	1.00	0.99	4.93	4.88
144	0.65	1.16	1.03	4.59	4.82
144	0.70	1.26	1.20	4.65	4.76
144	0.75	1.40	1.19	4.51	4.39
144	0.80	1.31	1.19	4.50	4.69
144	0.85	1.22	1.15	4.76	4.76
144	0.90	1.63	1.42	4.15	4.11
144	0.95	1.08	1.15	4.69	4.55

Note: IRI - International Roughness Index
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5 is good . New paving should be 4.4 or greater.

Faulting Data

North Bound Lane:

Chainage	Fault (in.)	
144.055	-0.19	
144.22	-0.17	PCCP Control
144.747	0.18	
144.937	0.14	Unjointed NMFRC Section

South Bound Lane:

Chainage	Fault (in.)	
144.927	0.11	
144.900	-0.16	Unjointed NMFRC Section
144.790	-0.10	
144.503	-0.20	
144.413	-0.12	Jointed NMFRC Section